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FOR DEVELOPING COUNTRIES

COMPENDIUM 13

Slopes: Analyses and Stabilization

Taludes: Análisis y estabilización

Talus: Analyses et stabilisation

prepared under contract AID/OTR-C-1591, project 931-1116,
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Notice

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Cover photo: Slide-prone areas cause maintenance problems in Bolivia.

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Foreword and Acknowledgments

This book is the fifteenth product of the Transportation Research Board's project on Transportation Technology Support for Developing Countries under the sponsorship of the U.S. Agency for International Development. The objective of this book is that it provide useful and practical information for those in developing countries who have direct responsibility for slope analysis and stabilization.
Feedback from correspondents in developing countries will be solicited and used to assess the degree to which this objective has been attained and to influence the nature of later products.

Acknowledgment is made to the following publishers for their kind permission to reprint the selected text portions of this compendium:
Central Road Research Institute, New Delhi; John Wiley and Sons, Inc., New York; U.S. Forest Service, Region 6, Portland, Oregon; and University of Tennessee, Engineering Experiment Station, Knoxville.

Prefacio y agradecimientos

Este libro es el décimoquinto producto del proyecto del Transportation Research Board sobre Apoyo de Tecnología de Transporte para Países en Desarrollo bajo el patrocinio de la U.S. Agency for International Development. El objetivo de este libro es el de proveer información útil y práctica para aquellos en países en desarrollo quienes tienen responsabilidad directa para el análisis y la estabilizacíon de suelos.
Se pedirá a los corresponsales en los países en desarrollo información sobre los resultados,para utilizarse en el asesoramiento del grado al cual se ha obtenido ese objectivo y para influenciar la naturaleza de productos subsequentes.

Se reconoce a los siguientes editores por el permiso dado para reimprimir las porciones de texto seleccionadas para este compendio:
Central Road Research Institute, New Delhi; John Wiley and Sons, Inc., New York; U.S. Forest Service, Region 6, Portland, Oregon; y University of Tennessee, Engineering Experiment Station, Knoxville.

Avant-propos et remerciements

Ce livre représente le quinzième volume du projet du Transportation Research Board sur la Technologie des transports à l'usage des pays en voie de développement. Ce projet est placé sous le patronage de l'U.S. Agency for International Development. L'objet de ce recueil est de réunir une documentation pratique et utile qui puisse aider les personnes responsables de l'analyse et de la stabilisation des sols. La réaction des correspondants des pays en voie de développement sera sollicitée et utilisée pour évaluer à quel point le but proposé de ce projet a été atteint, et pour influencer la nature des ouvrages à venir.

Nous remercions les éditeurs qui ont gracieusement donné leur permission de reproduire les textes sélectionnés pour ce recueil:
Central Road Research Institute, New Delhi; John Wiley and Sons, Inc., New York; U.S. Forest Service, Region 6, Portland, Oregon; et University of Tennessee, Engineering Experiment Station, Knoxville.

Overview

Background and Scope

Slope stabilization is the prevention of slope movement. Compendium 13 considers three types of slopes — natural slopes, cut slopes, and fill slopes.

Landslide is the term most commonly used to describe the failure of a natural slope. A landslide is the downward and outward movement of the slope-forming materials — natural rock, soils, artificial fills, or combinations of these materials. In fact, the term landslide is generic and is used to describe different types of movements, such as slides, flows, and falls, by various types of materials. The stabilization of slopes, either natural or man-made, first requires the identification of (a) the type of material involved, (b) the type of movement that has occurred or is anticipated, and (c) the cause(s) for the loss of stability that precedes actual failure.

The cut slope that remains after some existing material has been removed is the second type of slope considered. These slopes fail for the same technical reasons as do natural slopes. These failures can also be broadly termed landslides. Cut-slope failures are much more common than those of natural slopes because the physical act of removal of material tends to upset the equilibrium of the soil in its natural state.

The embankment or fill slope, the third type of

Vista General

Antecedentes y alcance

La estabilización de taludes es la prevención de sus movimientos. El Compendio 13 considera tres tipos de taludes — naturales, de corte y de relleno.

El término deslizamiento es el que comúnmente se utiliza para describir la falla de un talud natural. Un deslizamiento es el desplazamiento hacia abajo y hacia afuera de los materiales que forman el talud — piedra natural, suelos, rellenos artificiales, o combinaciones de las mismas. Se podría decir que el término deslizamiento es genérico, y que se utiliza para describir distintos tipos de desplazamientos, tales como deslizamientos, flujos, y caídas, por parte de varios tipos de materiales. En la estabilización de taludes naturales o hechos por el hombre, lo que primero se necesita es la identificación de (a) el tipo de material de que se trata, (b) el tipo de movimiento que ha ocurrido o que se anticipa, y (c) la causa o causas de la pérdida de estabilización que precede la falla misma.

Exposé

Historique et description

La stabilisation des talus est la prévention de leurs déplacements. Dans le recueil no. 13, trois sortes de talus sont examinées, les versants naturels, les déblais et les remblais.

"Glissement de terrain" est le terme couramment utilisé pour décrire la rupture d'un versant naturel. Un glissement de terrain est le mouvement vers le bas et vers l'extérieur des matériaux qui composent le versant — soient-ils des roches, des sols, des remblais de terrains d'apport, ou certaines combinaisons de ces matériaux. En fait, le terme "glissement de terrain" est un terme générique, et est utilisé pour la description de différentes sortes de mouvements: glissements, fluages et chûtes de différents types de matériaux. La stabilisation des talus, naturels ou artificiels, demande tout d'abord l'identification: (a) du type de matériau en question, (b) du type de mouvement qui a lieu ou qui est anticipé, et (c) de la ou les causes de la perte de stabilité qui précède la rupture même.

Le talus de déblai qui reste après que l'on a enlevé une certaine partie du matériau, est le second type de talus que nous allons considérer. Les mêmes raisons techniques sont la cause de la rupture de ces talus et de la rupture des versants naturels. Ces ruptures, d'une façon générale, peuvent être designées comme glissements de terrain. Les ruptures des talus de déblai sont beaucoup plus communes que celslope considered, is man-made. Under normal conditions (i.e., using suitable materials and proper placement and compaction of that material), fill slopes experience fewer failures than do cut slopes. Fills or embankments most commonly fail because of (a) erosion, (b) the natural drainage is blocked when an impermeable fill is constructed on a sidehill without underdrainage, (c) the soft material on which the fill has been constructed fails, displacing the fill downward, or (d) the fill itself starts to move down a natural slope. Fill-slope failures and fill failures are due to the same technical reasons as those for natural or cut slopes.

These same technical reasons contribute to the collapse of trench excavations (i.e., the walls fail) — either in fills, natural ground, or a combination of both.

The consideration of slope stability, therefore, encompasses all rock and soil found in or near a low-volume road. Gravity is the driving force for all landslides; all material is in a state of unstable equilibrium when it lies on a slope. An understanding of the basic mechanics of analyzing the materials at hand will assist the low-volume road engineer in determining what preventive or corrective measures are necessary to counter changes in soil equilibrium due to reductions in resistance values or increases in shearing stresses. This type of analysis should be used for proper route and materials selection and for determining proper construction methods. It will assure that the road is constructed at a minimum cost, or that the most economic solution will be found for slopes that fail during or after construction.

El segundo tipo de talud que se considera es el de corte, que queda después de que se haya quitado algún material. Estos taludes fallan por las mismas razones técnicas que los naturales. En términos generales estas fallas también pueden llamarse deslizamientos. Ocurren más a menudo que aquellos de taludes naturales, ya que la acción de quitar material tiende a trastornar el equilibrio del suelo en su estado natural.

El tercer tipo de talud a considerar, el de relleno, es hecho por el hombre. Bajo condiciones normales (es decir, si se utilizan materiales adecuados y se los coloca y compacta bien) tales taludes sufren menos fallas que los cortados. Generalmente fallan a causa de (a) erosión, (b) el drenaje natural obstruído por un relleno impermeable colocado sobre un talud de deslizamiento sin subdrenaje, (c) la falla del material blando sobre el que se construyó el terraplén desplazando éste hacia abajo, o (d) el relleno mismo, que empieza a deslizarse sobre un talud natural. Las fallas de taludes de relleno y de rellenos son debidas a las mismas razones técnicas que aquellas de taludes naturales o cortados.

Estas mismas razones técnicas contribuyen al colapso de excavaciones de zanjas (donde fallan las paredes) — en rellenos, suelo natural, o una combinación de ambos.

Por lo tanto, la consideración de la estabilidad de taludes incluye toda roca y suelo que se encuentre dentro o cerca de un camino de bajo volumen. La gravedad es la fuerza de impulso para todo deslizamiento; todo material se encuentra en estado de equilibrio inestable cuando está sobre una pendiente. Un entendimiento de la mecánica básica para analizar los materiales a mano ayudará al ingeniero de caminos de bajo volumen a determinar qué medidas correctivas o preventivas son

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les des versants naturels, car le simple acte d'enlever du matériau a tendance à bouleverser l'équilibre du sol dans son état naturel.

Le talus de remblai, le troisième genre de talus que nous allons examiner, est artificiel. Quand les conditions sont normales (c'est à dire quand on utilise des matériaux adéquats placés et compactés correctement), il y a moins de ruptures dans ces talus que dans les talus de déblai. Les ruptures de talus de remblai sont causées, la plupart du temps, par (a) l'érosion, (b) le drainage naturel qui est bloqué quand on construit un remblai imperméable sur un versant de colline sans drainage souterrain, (c) il y a rupture du matériau mou sur lequel on a construit le remblai, ce qui cause le glissement par le bas de celui-ci, ou (d) le remblai lui-même commence à glisser le long d'une pente naturelle. Les ruptures des talus de remblai, et des remblais eux-mêmes, sont dûes aux mêmes conditions techniques que celles des talus de déblai et des versants naturels.

Ces mêmes raisons contribuent à l'effondrement des tranchées (les murs s'effondrent) soit de remblais, de sol naturel, ou de la combinaison des deux.

L'étude de la stabilité des talus donc inclut tous les sols et roches qui se trouvent sur une route économique, ou à ses alentours. La gravité est la force dominante de tous les glissements de terrain; le matériau est dans un état d'équilibre instable quand il est placé sur une pente. La

Water is one of the major causes of slope failures. The entire subject of erosion control is an integral part of slope stabilization as are many facets of drainage design. Texts that are related to the problem of slope stabilization appear in ***Compendium 2, Drainage and Geological Considerations in Highway Location; Compendium 3, Small Drainage Structures; Compendium 5, Roadside Drainage; Compendium 6, Investigation and Development of Material Resources;*** and ***Compendium 9, Control of Erosion.*** Thus, Compendium 13 stresses other slope stability problems and solutions rather than detailed methods of preventing surface erosion. Although this compendium does not attempt to repeat soil information introduced in previous compendiums, such information is also critical to the analysis of slope stability.

This compendium is directed to the general highway engineer. Slope movement is a complex phenomenon that is often poorly understood by the nonspecialist. The texts were selected to present the basic principles of slope stabilization and to provide general approaches to the correction of the more common slope stabilization problems. The proper solution for complex stabilization problems requires a depth of knowledge and experience that is beyond the scope of this compendium. The prudent highway engineer will call on an expert for assis-

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necesarias para combatir cambios en el equilibrio del suelo, debidos a reducciones en los valores de resistencia o aumentos en el esfuerzo cortante. Este tipo de análisis se deberá utilizar para una correcta ubicación del trazado y selección de los materiales y para determinar el método correcto de construcción. Asegurará que el camino se construirá a costo mínimo, o que se podrá encontrar la solución más económica para los taludes que fallan durante o después de la construcción.

Una de las causas principales de falla es el agua. El tema completo de control de la erosión es una parte íntegra de la estabilización de taludes, al igual que muchos de los aspectos del diseño de drenaje. Por lo tanto, el ***Compendio 2: Consideraciones de drenaje y geológicas en la ubicación del camino***, el ***Compendio 3: Pequeñas estructuras de drenaje***, el ***Compendio 5: Drenaje del borde de la carretera***, el ***Compendio 6: Investigación y desarrollo de recursos de materiales***, y ***Compendio 9: Control de erosión***, contienen textos que se relacionan con el problema de estabilización de taludes. Por lo tanto, el Compendio 13 subraya otros problemas y soluciones en la estabilización de taludes, en vez de presentar métodos detallados para la prevención de la erosión de la superficie. Aunque este compendio no intenta repetir la información sobre suelos presentada en compendios previos, esta información también es importante para el análisis de la estabilización de taludes.

Este compendio se dirige al ingeniero vial general. El desplazamiento de taludes es un fenómeno complejo y muchas veces el que no

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compréhension du mécanisme de base de l'analyse des matériaux qu'il a sous la main, aidera l'ingénieur routier de routes à faible capacité à déterminer quelles actions, preventives ou correctives, sont nécessaires pour parer aux changements d'équilibre du sol dûs aux réductions des valeurs de résistance, ou aux augmentations des tensions de cisaillement. Ce genre d'analyse devrait être utilisé pour le choix du tracé de la route et des matériaux, et pour la détermination de méthodes de construction correctes. On aura ainsi l'assurance que la route sera construite à un coût minimal, ou qu'on trouvera la solution la plus économique à la rupture des talus, pendant ou après la construction.

L'eau est l'une des raisons majeures de rupture de talus. Tout ce qui comprend le contrôle de l'érosion et beaucoup d'aspects du drainage, font partie intégrale de la stabilisation des talus. C'est pourquoi les recueils no. 2, ***Considérations sur les facteurs de drainage et de géologie qui influencent le choix de l'emplacement d'une route***; no. 3, ***Petits ouvrages de drainage***; no. 5, ***Drainage des bas-côtés de la route***; no. 6, ***Investigation et développement des gisements de matériaux routiers***; et no. 9, ***Contrôle de l'érosion*** contiennent des textes qui sont apparentés au problème de la stabilisation des talus. Dans ce recueil no. 13, on porte une attention spéciale à d'autres problèmes de stabilité des talus et à leurs solutions, plutôt que de s'étendre en détail sur les méthodes de prévention de l'érosion de surface. Dans ce recueil nous n'allons pas non plus essayer d'enseigner de nouveau certains aspects de la pédologie, ainsi que nous l'avons fait dans des recueils précédents, mais nous insistons sur le fait qu'une bonne connaissance de ceux-ci est cruciale pour analyser correctement la stabilité des talus.

Ce recueil est écrit à l'intention de l'ingénieur routier généraliste. Le déplacement des talus est un phénomène très complexe, qui n'est souvent tance when analyzing slope stabilization problems that are unusual, expensive, or dangerous to human life.

Rationale for This Compendium

Slope stabilization is a problem that is inherent throughout all phases of highway work. The solution to slope stabilization problems begins with the ability to identify the types of materials involved and the associated modes of failure that can occur in these materials. Once the mechanics of evaluating slope stabilization problems are understood, the solution of possible or actual individual slope failures becomes a matter of economics. Slope stability evaluations should take place not only during preliminary road location but also during actual design, during construction of the road, and, as necessary, during the life of the road.

The first evaluation of slope stabilization problems should occur as part of the initial road location decision. (See Compendium 2, Selected Text 9—*Landslide Investigations: A Field Handbook for Use in Highway Location and Design.*) It is in this phase that the economics of avoidance of unstable areas is most attractive. Avoidance may also be the most economic alternative during later phases (i.e., during design or because of failures during or after construction). However, the highest economic benefit will obviously occur if the avoidance solution is evaluated and selected as soon as possible.

The second evaluation of slope stabilization problems should occur during the detailed design of the road. At this time, localized slope stability evaluations can be made. The proper side slopes can be determined for the materials on site, and control measures such as retaining walls can be most economically planned. Note that further economic benefits can be realized during this design evaluation period by the

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sea especialista no lo comprende perfectamente. Los textos fueron seleccionados para presentar los principios básicos de estabilización de taludes y para proveer métodos generales para la corrección de los problemas más comunes de estabilización. La correcta solución para los problemas de estabilización más complejos requiere un conocimiento profundo y experiencia extensa que están más allá del alcance de este compendio. El ingeniero vial prudente, al enfrentarse con el análisis de un problema de estabilización que es caro, peligroso, o poco común, consultará con un experto en la materia.

Exposición razonada para este compendio

La estabilización de taludes es un problema inherente a todas las etapas de trabajo vial. La solución para estos problemas comienza con la habilidad de identificar los tipos de materiales involucrados y los modos asociados de falla que los acompañan. Una vez que se ha compren-

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pas très bien compris de ceux qui n'ont pas reçu de specialisation en ce sujet. Nos textes ont été choisis pour présenter les principes de base de la stabilisation des talus, et pour fournir une approche générale à la correction des problèmes les plus communément rencontrés. La solution correcte de problèmes complexes de stabilisation demande une connaissance profonde de ce sujet, conjuguée avec une longue expérience pratique—et ceci dépasse l'envergure de notre recueil. L'ingénieur routier prudent demandera l'aide d'un expert en la matière s'il doit analyser des problèmes de stabilisation de talus qui sont inhabituels, très onéreux, ou qui risquent de metre en danger des vies humaines.

Objectif de ce recueil

La stabilisation des talus est un problème qui se pose à toutes les phases de la construction routière. Pour pouvoir résoudre ce problème il faut d'abord être capable d'identifier les types de matériaux avec lesquels on va devoir travailler et leurs modes de rupture. Une fois que l'on a bien saisi la technique d'évaluation des problèmes de stabilisation des talus, la résolution de ces problèmes, potentiels ou actuels, devient une question économique. Les évaluations de la stabilité des talus devraient être envisagées, non seulement au stade du tracé préliminaire de la route, mais aussi aux stades de la conception et de la construction et, le cas écheant, pendant toute la vie de la route.

La première évaluation du problème devrait prendre place au moment où l'on va décider le tracé initial de la route (voir Recueil no. 2, Texte choisi 9: Etudes de glissements de terrain: Un manuel pour le dimensionnement et l'emplacement des routes). C'est à ce stage que la décision d'éviter les endroits instables est la plus proper determination of side slope angles based on stability criteria rather than on the acceptance of arbitrary "standard" side slopes often indicated on "typical cross sections."

The third evaluation of slope stabilization problems should occur during the construction phase. Material types may prove to be different than anticipated, and signs of seasonal underground water flows or fluctuating water tables may be uncovered. These conditions should be evaluated as soon as they are identified because the cost to solve such potential slope stability problems before they occur will be much less than that to correct subsequent failures.

The fourth evaluation of slope stabilization problems occurs after construction is finished—when either signs of distress appear or a slope actually fails. Again, it is more economic and safer to solve these problems when the first signs appear rather than after the subsequent failure.

The evaluation of slope stability is made by determining the factor of safety of a slope (i.e., the ratio of the shearing resistance to the weight of the sliding mass). The factor of safety is 1 just before slope failure. Slopes may therefore be stabilized by increasing shearing resistance or reducing shear stress. Most cut slopes are least stable immediately after the cut is made but may fail at any time if moisture conditions become more severe than anticipated. Fill slopes normally fail quite a while after the fill has been

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dido la mecánica de evaluación de los problemas de estabilización, la solución de fallas individuales, potenciales o actuales, se vuelve asunto de costo. No sólo se deberán llevar a cabo evaluaciones de estabilidad de taludes durante la ubicación preliminar del camino, sino también durante el diseño actual, durante la construcción del camino, y cuando sea necesario durante la vida útil del camino.

La primera evaluación de problemas de estabilización deberá ocurrir como parte de la decisión inicial de ubicación del camino (véase Compendio 2, Texto Seleccionado 9, ***Investigaciones de derrumbes: Un manual de campaña para uso en el diseño y ubicación de carreteras***). Es en esta etapa que la sensatez económica de evitar áreas inestables es más atractiva. Evitación también podría ser la alternativa más económica durante fases posteriores (es decir, durante el diseño o por fallas durante o después de la construcción). Es obvio que el beneficio económico más grande ocurrirá cuando la solución de evitación se evalúa y selecciona lo más pronto posible.

La segunda evaluación de los problemas de estabilización de taludes deberá ocurrir durante el diseño detallado del camino. Es en esta etapa que se pueden llevar a cabo las evaluaciones locales de estabilidad de taludes. Se pueden determinar los correctos taludes laterales para los materiales en situ, y las medidas de control, tales como muros de contención, pueden ser planeadas en forma más económica. Deberá notarse que se pueden realizar más beneficios

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attrayante du point de vue économique. Cette décision peut être aussi l'alternative la plus économique durant les phases ultérieures du projet (conception, dimensionnement, ou au moment de ruptures pendant ou après la construction. Cependant le plus tôt on evalue le problème et la décision est prise, le plus important sera l'avantage économique.

La seconde évaluation du problème devra prendre place au moment du dimensionnement detaillé de la route. C'est en effet à cette période que l'on peut évaluer localement la stabilité des talus. Les pentes correctes peuvent être déterminées en fonction des matériaux en place, et des mesures de contrôle, telles que les murs de soutènement par exemple, peuvent être projetées beaucoup plus économiquement. Il faut bien remarquer aussi que le prix de revient peut être encore plus diminué si l'on a soin, pendant cette évaluation, de calculer les angles des pentes en se basant sur les critères de stabilité, plutôt qu'en acceptant des "normes" arbitraires basées sur des "sections transversales typiques".

La troisième évaluation des problèmes de stabilisation des talus devrait avoir lieu lors de la construction elle-même. Les matériaux peuvent être différents de ceux qui étaient anticipés, et l'on peut découvrir des marques indiquant la présence de courants saissonniers d'eau souterraine ou de fluctuations de la nappe phréatique. On devrait évaluer ces conditions aussitôt que possible après leur identification, car il est beaucoup moins onéreux de résoudre ces problèmes potentiels avant que le dommage ne soit fait, que de réparer les ruptures.

La quatrième évaluation des problèmes de stabilité des talus, a lieu quand la construction est terminée—s'il est apparent qu'il va y avoir rupture ou s'il y a rupture. De nouveau, il est beaucoup plus économique de résoudre ces problèmes dès l'apparence des premiers completed unless the cause is foundation failure, in which case the fill subsides soon after it is completed. Fill slopes normally fail from their own weight rather than from the weight of the vehicles using the road.

Slope stabilization problems in engineering soils can be divided into two basic types, depending on the influence of the height of the slope on its stability. Cohesionless material depends on mechanical strength that increases with the normal component of the weight (one reason for the compaction of granular fills). Therefore, any slope of cohesionless material that is at a flatter angle than the angle of internal friction will be stable regardless of the height of the slope. However, if high ground water is present in coarse-grained material, one-half the angle of internal friction is normally considered as the stability limit. Cohesive soil, on the other hand, depends on attraction between soil particles and moisture, which are independent of the soil's weight. Stability in cohesive soil is dependent mainly on the steepness and height of the slope and the shear strength of the soil. Cohesive soil may suffer (a) slope failure where the bottom of the curved failure plane is located at or above the toe of the slope or (b) base failure where the face of the curved failure plane is tangent to some less-yielding surface below the toe of the slope, in which case the intersection of the failure plane with the surface occurs beyond the toe of the slope. (These types of failure are illustrated on Selected Text page 36.) Slopes in mixed soils, i.e., soils that are neither purely cohesionless nor totally cohesive such as some mixed clays and gravels, are also dependent on

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económicos durante este período de evaluación del diseño, determinando correctamente los ángulos de talud lateral basados en criterios de estabilidad, en vez de taludes laterales "norma" recomendados a menudo en "secciones transversales típicas".

La tercera evaluación de problemas de estabilización de talud se deberá llevar a cabo durante la etapa de construcción. Podrá ocurrir que los tipos de materiales son distintos de lo anticipado, y que se descubren flujos estacionales de agua subterránea o fluctuaciones del nivel freático. Estas condiciones deberán evaluarse tan pronto como se identifiquen porque el costo de resolver tales problemas potenciales de estabilidad de talud antes que ocurran sería mucho menos que el de corregir la falla subsecuente.

La cuarta evaluación de problemas de estabilización de talud ocurre después de terminarse la construcción — cuando aparecen señales de tensión o realmente falla el talud. Se reitera que es más ecónomico y seguro resolver estos problemas cuando aparezcan las primeras señales en lugar de después de la falla subsecuente.

Se hace la evaluación de estabilidad determinando el factor de seguridad de un talud (es decir, la razón de la resistencia al esfuerzo cortante al peso de la masa que se desliza). El factor de seguridad es 1 en el momento antes de la falla del talud. Por lo tanto los taludes pueden estabilizarse aumentando la resistencia al esfuerzo cortante o reduciendo el esfuerzo cortante. El período de más inestabilidad para casi todo talud es inmediatamente después de que se haya realizado el corte, pero puede fallar en

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signes, plutôt que de différer les réparations jusqu'à la rupture complète.

L'évaluation de la stabilité des talus se calcule en déterminant le coéfficient de sécurité d'une pente (c'est à dire le rapport de la résistance au cisaillement au poid de la masse interessée par le glissement). Le facteur de sécurité est égal à 1 juste avant la rupture. On peut donc stabiliser les pentes soit en augmentant la résistance au cisaillement, soit en réduisant la tension de cisaillement. La plupart des talus de déblai sont stables juste après le déblaiement mais peuvent se rompre d'un moment à l'autre si les conditions d'humidité deviennent plus sévères que l'on avait anticipé. Les talus de remblai, d'ordinaire, n'ont tendance à la rupture qu'après un certain temps, sauf si la cause en est la rupture de la fondation et, dans ce cas, le remblai s'affaisse très tôt après qu'il soit construit. Normalement, la rupture des talus de remblai est causée par leur propre poids, plutôt que par le poids des véhicules circulants sur la route. Il y a fondamentalement deux sortes de problèmes de stabilisation de talus en sols routiers, selon l'influence de la hauteur du talus sur sa stabilité. Un sol sans cohésion dépend de la résistance mécanique qui augmente avec la composante normale du poids (une des raisons pour lesquelles on compacte les remblais en sols granuleux). Donc tout talus en sol non-cohérent qui est à un angle plus plat que l'angle de frottement interne sera stable quelle que soit sa hauteur. Toutefois si dans un materiau grossier l'eau phréatique est élevée, la moitié de l'angle de frottement interne est normalement considérée comme la limite de stabilité. Les sols cohérents, the steepness and height of the slope.

Both surface water and ground water are important factors in slope stability, not only because of the surface erosion problems discussed in previous texts but also because of (a) internal erosion problems (i.e., piping), (b) the hydrostatic pressure introduced within the soil mass itself that reduces shear strength, (c) the additional weight of the water in the soil mass that increases shear stress, and (d) the reduction in strength of cohesive soils with increasing moisture content.

Slope stability analyses are best made by specialists because there are so many unknowns involved that require subjective evaluations. In many cases, however, the availability or the cost of such specialists makes their use impractical in the location, design, and construction of low-volume roads. Although this compendium presents information that should be of assistance to the general highway engineer, some of the more theoretical aspects of slope stabilization have been omitted to allow the inclusion of some generalized, simplified approaches to a complex problem. Whenever an improper slope stabilization analysis may lead to the possibility of the loss of life or high economic losses to the community, a specialist should be consulted.

Discussion of Selected Texts

The first text, *Chapter 2 — Slope Movement Types and Processes*, is excerpted from *Landslides: Analysis and Control* (Special Report 176, Transportation Research Board, 1978). It reviews a fairly complete range of slope-movement processes. It identifies and classifies them according to features that are also to some

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cualquier momento si las condiciones de humedad se vuelven más graves de lo anticipado. Los taludes de relleno normalmente fallan un tiempo después de que se haya completado el relleno, al menos que haya una falla por la base, en cuyo caso el relleno se hunde poco después de completarse. Estos taludes normalmente fallan por razón de su propio peso, en vez de por el peso de los vehículos que utilizan el camino.

Los problemas de estabilización en suelos ingenieriles pueden dividirse en dos tipos básicos, dependiendo de la influencia de la altura del talud sobre su estabilidad. El material no cohesivo depende de resistencia mecánica que aumenta con el componente normal del peso (una de las razones para la compactación de rellenos granulares). Por lo tanto, un talud de material no cohesivo es estable, cualquiera sea su altura, siempre que el ángulo del talud sea menor que el ángulo de fricción interna. Sin embargo, si hay agua freática cerca de la superficie en material de granulación gruesa, la mitad del ángulo de fricción interna normalmente se considera como el límite de estabilidad. Por otra parte, el suelo cohesivo depende de la atracción entre las partículas de suelo y la humedad, que son independientes del peso del suelo. Entonces la estabilidad del suelo cohesivo depende principalmente de la pendiente y altura del talud y la resistencia al esfuerzo cortante del suelo. En suelos cohesivos, puede producirse (a) falla de talud donde la parte inferior del plano curvo de deslizamiento se encuentra al pie del talud o más arriba o (b) falla por la base donde la cara del plano curvo de deslizamiento es tangente con una superficie menos movediza que se encuentra debajo del pie del talud, en cuyo caso la intersección del plano de deslizamiento con la superficie ocurre más allá de dicho pie. Estos tipos de falla de talud han sido ilustrados en la página 36 de los textos seleccionados. Los taludes en suelos mixtos, es de-

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d'un autre côté, dépendent de l'attraction entre les particules de sol et l'humidité, indépendamment du poids du sol. La stabilité des sols cohérents est fonction principalement de la hauteur et de l'inclinaison de la pente, et de la résistance au cisaillement du sol. Dans un sol cohérent on peut avoir: (a) rupture du talus quand la base de la surface courbe de glissement est située au pied ou au dessus du pied du talus; ou (b) rupture par la base, quand la surface courbe de glissement est tangente à une surface résistante au dessous du pied du talus, dans ce cas la surface de glissement coupe le talus au-delà de son pied. Ces sortes de rupture sont illustrées dans les textes choisis, à la page 36. Les talus en sols mixtes, c'est à dire en sols qui ne sont pas complètement non-cohérents, ni complètement cohérents, comme certaines argiles et certains granulats, dépendent aussi de la pente et de la hauteur du talus.

L'eau phréatique et l'eau de surface sont des facteurs importants de la stabilité des talus, non seulement à cause des problèmes d'érosion superficielle que nous avons discutés dans des textes précédents, mais aussi à cause de: (a) problèmes d'érosion interne (renards); (b) la pression hydrostatique introduite dans la masse de sol, qui abaisse la résistance au cisaillement, (c) le poids additionnel de l'eau dans la masse de sol, qui augmente la contrainte de cisailledegree relevant to their recognition, avoidance, control, or correction. The chief criterion used in the classification is type of movement. The type of material is a secondary criterion. The types of movement include (a) falls; (b) topples; (c) slides, either rotational or translational; (d) spreads; (e) flows; and (f) complex slope movements that include combinations of two or more of the first five types. Materials are divided into two classes — rock and engineering soil. Soil is further divided into debris and earth. (Some of the various combinations of movements and materials are shown in Figure 2.1, which is included as an insert with this compendium.)

This text also introduces the causes of sliding slope movements (slope instability). These causes include both factors that contribute to an

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cir, suelos que no son puramente no cohesivos ni totalmente cohesivos, tales como algunas arcillas y gravas mixtas, también dependen de la pendiente y altura del talud.

El agua de superficie y el agua subterránea son factores importantes en la estabilidad de taludes, no sólo por los problemas de erosión de la superficie sobre los que se habló en textos previos, sino también por (a) problemas de erosión interna (es decir, tubificación), (b) la presión hidrostática introducida dentro de la masa de suelo misma que reduce la resistencia al esfuerzo cortante, (c) el peso adicional del agua en la masa del suelo que aumenta el esfuerzo cortante, y (d) la disminución de la resistencia en suelos cohesivos cuando va aumentando el contenido de humedad.

Es mejor que un especialista realice el análisis de estabilidad de taludes, ya que existen muchas incógnitas que requieren evaluaciones subjetivas. Sin embargo, hay muchos casos donde la disponibilidad o costo de tal especialista hace impracticable su participación en la ubicación, diseño, y construcción de caminos de bajo volumen. Aunque este compendio presenta información que puede ayudar al ingeniero vial general, se han omitido algunos de los aspectos más teóricos de estabilización para permitir que se incluyan algunos métodos generalizados y simplificados para resolver un problema complejo. Cuando un análisis incorrecto de estabilización de talud puede provocar posibles pérdidas de vida o de grandes cantidades de dinero para la comunidad, se deberá consultar con un especialista.

Presentación de los textos seleccionados

El primer texto, *Chapter 2 — Slope Movement Types and Processes* (Capítulo 2 — Tipos y procesos de movimiento de taludes), fue extraído de *Landslides: Analysis and Control* (Deslizamientos: análisis y control, Special Report 176, Transportation Research Board, 1978). Repasa una gama bastante completa de procesos de movimiento de taludes. Los identifica y clasifica

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ment et (d) la moindre résistance des sols cohérents à mesure que leur teneur en eau augmente.

On devrait laisser aux spécialistes le soin d'analyser la stabilité des talus, car ces analyses demandent une évaluation subjective d'un bon nombre d'inconnues. En maintes occasions toutefois, on est obligé de se passer de leur expertise, car ils ne sont pas disponibles et leur emploi est trop onéreux pour être justifié dans la conception, location et construction de routes économiques. Bien que dans ce recueil nous présentons un ensemble de documents qui devrait être utile à l'ingénieur routier généraliste, nous avons omis certains des côtés les plus théoriques de la stabilisation des talus, afin de nous permettre d'inclure quelques solutions simples et générales à ce problème complexe. Mais aussitôt qu'il y a le moindre risque qu'une analyse incorrecte de la stabilité d'un talus puisse mettre en péril une vie humaine, ou même causer de gros dommages économiques à la communauté, on devrait consulter un expert.

Discussion des textes choisis

Le premier texte est le deuxième chapitre, *Slope Movement Types and Processes* (Types et développement de mouvements de talus) du rapport *Landslides: Analysis and Control* (Glissements de terrain: Analyse et contrôle, Special Report 176, Transportation Research Board, 1978). On y passe en revue une grande variété de mouvements de terrain et le processus de leur évolution. On les identifie et on les classe selon les éléments caractéristiques qui permettent de les identifier, les éviter, les contrôler ou les corriger. Le critère principal de cette classification est le type de mouvement de terrain. Le genre de matériau est le critère secondaire. Les types de mouvements de terrain sont: (a) les chûtes, (b) les basculements, (c) les glissements rotationnels ou translationnels, (d) les increased shear stress and factors that contribute to low or reduced shear strength. The analyses of slope failures and corrective measures necessary to prevent or repair these failures (i.e., slope stabilization), which are included in the other texts, are all based on (a) reducing shear stress along a potential plane of failure or (b) increasing shear strength in the material subject to slope movement.

The second text, *Art. 35, Stability of Slopes*, is excerpted from *Soil Mechanics in Engineering Practice* (2nd ed., John Wiley and Sons, Inc., 1967). It introduces the concept that slope failure occurs at that time when the shear stress caused by the weight of the sliding mass overcomes the shear strength or resistance to sliding of the material involved. Just prior to failure, the weight and shearing resistance are equal. The ratio of the shearing resistance to the weight of the sliding mass is termed the factor of safety F; therefore, at the instant before failure, F = 1. A slope is considered stable when F is determined to be greater than 1 and unstable when F is less than 1.

Slopes of cohesionless material depend on internal friction for stability. The angle of internal

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de acuerdo con características que también a cierto grado son aplicables a su reconocimiento, evasión, control, o corrección. El criterio principal utilizado en la clasificación es el tipo de movimiento. El tipo de material es un criterio secundario. Los tipos de movimiento incluyen (a) caída; (b) derribo, (c) deslizamiento, rotatorio o de traslación; (d) desparramo; (e) flujo; y (f) movimientos de talud complejos que incluyen combinaciones de dos o más de los primeros cinco tipos. Los materiales se dividen en dos clases — roca y suelo ingenieril. Además el suelo se divide en detritos y tierra. (Se presentan algunas de las diversas combinaciones de movimientos y materiales en la Figura 2.1 incluída en este compendio.)

Este texto también habla sobre las causas de los movimientos de deslizamiento de taludes (inestabilidad de taludes). Estas causas incluyen factores que aumentan la acción del esfuerzo cortante y factores que disminuyen la resistencia al esfuerzo cortante. Los análisis de fallas de taludes y las medidas correctivas necesarias para impedir o reparar estas fallas (es decir, la estabilización de taludes), incluídos en otros textos, todos se basan en (a) reducción del esfuerzo cortante sobre el plano de deslizamiento potencial, o (b) aumento de la resistencia al esfuerzo cortante en el material sujeto al movimiento del talud.

El segundo texto, *Art. 35, Stability of Slopes* (Art. 35, Estabilidad de taludes), fue extraído de *Soil Mechanics in Engineering Practice* (Mecánica de suelos en la ingeniería práctica, 2d. ed., John Wiley and Sons, Inc., 1967). Propone el concepto de que la falla del talud ocurre en el momento cuando el esfuerzo cortante producido por el peso de la masa en deslizamiento vence la resistencia al esfuerzo cortante, es decir al deslizamiento del material involucrado. Justo antes del deslizamiento el peso y la resistencia al esfuerzo cortante son iguales. La razón de la resistencia al esfuerzo cortante al peso de la masa en deslizamiento se llama el factor de seguridad F; por lo tanto, en el momento antes de la falla, F = 1. Un talud se considera estable cuando se determina que F es mayor que 1 e inestable cuando F es menor que 1.

étalements; (e) le fluage; et (f) les mouvements complexes qui comprennent des combinaisons de deux ou plus des cinq premiers désordres décrits ci-dessus. Les matériaux sont divisés en deux classes — roches et sols routiers. Les sols sont subdivisés en débris et terre. (Quelques combinaisons de mouvements et de matériaux sont illustrées dans la figure 2.1 de l'encart de ce recueil).

Les causes des mouvements glissants des pentes (instabilité des talus) sont introduites. Ces causes comprennent les facteurs contribuants à une augmentation de la tension de cisaillement, et ceux qui contribuent à une résistance au cisaillement basse ou réduite. Les analyses des ruptures de talus et des mesures correctives nécessaires à la prévention ou la réparation de ces ruptures (c'est à dire la stabilisation des talus), qui sont incluses dans les autres textes sont toutes basées sur (a) la réduction de la tension de cisaillement sur une surface de glissement potentielle ou (b) l'augmentation de la résistance au cisaillement du matériau sujet au mouvement de talus.

Le deuxième texte, *Art. 35, Stability of Slopes* (Stabilité des talus), est extrait de *Soil Mechanics in Engineering Practice* (Mécanique des sols appliquée, 2^{ieme} édition, John Wiley and Sons, Inc., 1967). On introduit le concept que la rupture d'un talus se produit au moment précis ou la tension de cisaillement causée par le poids de la masse interessée par le glissement l'emporte sur la résistance au cisaillement, ou résistance au glissement du matériau dont il est question. Juste avant la rupture le poids et la résistance au cisaillement sont égaux. Le rapport friction ϕ is the angle above the horizontal at which the loose cohesionless material will just stand. (This is the angle at which F = 1.) For any given slope in cohesionless material, the factor of safety is found by dividing the tangent of the angle of internal friction by the tangent of β, the angle of the slope under investigation (the angle being measured from a horizontal plane). The height of slopes in cohesionless soil is not a factor in the stability of the slope.

The stability of slopes in cohesive soils is more complicated and is based on Rankine's earth-pressure theory. In general, a homogeneous cohesive soil will fail along a curved surface. The text describes the general character of slides in homogeneous cohesive soil. The mathematical justification for this is not excerpted for this compendium; the final information is reduced to charts that are included in the text. The center of the curved surface along which a homogeneous cohesive soil will fail can be located from information shown on the charts. By using the slope angle β and the relation between the slope height and depth to a firm base beneath the cohesive soil, the type of possible failure (slope or base) and the location of the center of the critical circle for slope failure can be graphically located. The safety of slopes in cohesive soils is

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Los taludes compuestos de material no cohesivo dependen de la fricción interna para su estabilidad. El ángulo de fricción interna ϕ es el ángulo sobre el horizontal en el que el material suelto, no cohesivo puede soportarse. (Este es el ángulo donde F = 1.) Dado cualquier talud de material no cohesivo, se encuentra el factor de seguridad dividiendo la tangente del ángulo de fricción interna por la tangente de β, el ángulo del talud que se está investigando (se deberá medir este ángulo en un plano horizontal). La altura de los taludes de suelo no cohesivo no es un factor en su estabilidad.

La estabilidad de taludes en suelos cohesivos es más complicada y se basa en la teoría de Rankine del empuje de tierras. Por lo general, un suelo homogéneo cohesivo fallará sobre una superficie curva. El texto describe las características generales de los deslizamientos en este tipo de suelo. No se ha incluído en el compendio su justificación matemática; la información final se ha reducido a diagramas que se han incluído en el texto. Se puede localizar el centro de la superficie curva por donde fallará un suelo homogéneo cohesivo, utilizando la información presentada en los diagramas. Se puede averiguar por medio de gráficos el tipo de falla potencial (de talud o por la base) y el centro del círculo crítico de falla de talud, si se utiliza el ángulo de talud β y la relación entre la altura del talud y el espesor hasta una base firme debajo del suelo cohesivo. La seguridad de los taludes en suelos cohesivos depende de la pendiente y altura.

El texto también evalúa los taludes de suelos con cohesión y fricción interna. La seguridad de los taludes en dichos suelos también depende de su pendiente y altura.

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de la résistance au cisaillement au poids de la masse interessée par le glissement, est appelé le coefficient de sécurité F; donc, à l'instant avant la rupture, F = 1. Un talus est considéré stable quand F est plus grand que 1, et instable quand F est plus petit que 1.

Les talus construits en matériaux non-cohérents dépendent du frottement interne pour leur stabilité. L'angle de frottement interne ϕ est l'angle au dessus de l'horizontale auquel le matériaux non-cohérent et non-compacté reste stationnaire (l'angle auquel F = 1). Pour trouver le coéfficient de sécurité d'un talus de matériau non-cohérent, on divise la tangente de l'angle de frottement interne par la tangente de β, l'angle du talus en question (l'angle que forme le talus avec l'horizontale). La hauteur d'un talus en matériau non-cohérent n'a aucun effet sur sa stabilité.

La stabilité des talus de matériau cohérent est plus compliquée, et est basée sur la théorie de la poussée des terres de Rankine. En général la rupture d'un matériau cohérent homogène sera le long d'une surface courbe. Dans le texte on décrit les caractéristiques générales des glissements de sol cohérent homogène. La justification mathématique de ceci n'est pas incluse dans le texte reproduit pour ce recueil; nous avons inclus le résultat final, en forme de tables de calcul. Le centre de la surface courbe de rupture d'un sol cohérent homogène peut être déterminé en se servant des tables de calcul. En utilisant l'angle du talus β et le rapport entre la hauteur et la profondeur du talus et une base résistante au dessous du sol cohérent, le type de rupture potentielle (par la base ou de talus) et le centre du cercle critique, peuvent être calculés graphiquement. La sécurité des talus de sols cohérents dépend de leur inclinaison et de leur hauteur.

On évalue aussi les talus de sols cohérents à frottement interne. La sécurité des talus dans dependent on both steepness and height.

The text also evaluates slopes on soils with both cohesion and internal friction. The safety of slopes on these soils is also dependent on steepness and height. A chart solution that uses the slope angle β and the angle of internal friction ϕ is presented. Failures in soils with both cohesion and internal friction will occur along toe circles unless ϕ is smaller than approximately 3°. A further description of the use of the charts mentioned above is included in the next selected text.

A method of investigating irregular slopes on nonuniform soil is described in the text as is the investigation of a composite surface of sliding (a noncircular slip plane). Both these investigations use the method of slices. This text presents the theory of the method of slices. A practical solution using the method of slices is included in the fifth selected text.

The third text, ***Chart Solutions for Analysis of Earth Slopes***, appeared in ***Highway Research Record 345*** (Highway Research Board, 1971). It compiles practical chart solutions for the slope stability problem and is concerned with the use of the solutions rather than with their derivations. The previous text described the development for the figures presented under the Taylor Solution section of this text. Also included in this paper are (a) the Bishop and Morgenstern Solution, which is based on an adaptation of the Swedish slice method; (b) the Morgenstern Solution, which can be used for highway embankments that at times act as dams; (c) the Spencer Solution, which is a more generalized solution of Bishop's adaptation of the Swedish slice method; (d) the Hunter Solution, which accounts for variations in the water table; and (e) the Hunter and Schuster Solution, which is a special case of the Hunter Solution.

Se presenta una solución en diagrama que utiliza el ángulo de talud β y el ángulo de fricción interna ϕ. Las fallas en los suelos cohesivos y con fricción interna ocurrirán a lo largo de los círculos que pasan por el pie de talud al menos que ϕ sea menor que aproximadamente 3°. En el próximo texto seleccionado se han incluído más explicaciones del uso de los diagramas mencionados arriba.

Se describe en el texto un método de investigar los taludes irregulares en suelo no uniforme, como también la investigación de una superficie de deslizamiento compuesta (un plano de deslizamiento no circular). Ambas investigaciones utilizan el método de tajadas, la teoría del cual es presentada en este texto. Una solución práctica utilizando este método se incluye en el quinto texto seleccionado.

El tercer texto, ***Chart Solutions for Analysis of Earth Slopes*** (Soluciones gráficas para el análisis de taludes de tierra), apareció en el ***Highway Research Record 345*** (Registro de investigación vial 345, Highway Research Board, 1971). Compila soluciones factibles en diagrama para el problema de estabilidad de talud y se concierne con el uso de las soluciones, más que con sus derivaciones. El texto previo describe como se desarrollaron las figuras presentadas en la sección de la Solución de Taylor en este texto. También se incluyen en este artículo (a) la Solución de Bishop y Morgenstern, que se basa en una adaptación del método sueco de tajadas; (b) la Solución de Morgenstern, que se puede utilizar para los terraplenes de carreteras que a veces actúan como presas; (c) la Solución de Spencer, que

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ces sols dépend aussi de leur hauteur et de leur inclinaison. Une solution par table de calcul, qui utilise l'angle du talus β et l'angle de frottement interne ϕ est présentée. La rupture des matériaux décrits ci-dessus se produira au pied du talus, à moins que ϕ soit plus petit qu'à peu près 3°. Une description plus élaborée de l'emploi des tables de calcul dont nous venons de faire mention, est incluse dans le texte choisi suivant.

Une méthode d'investigation des talus irréguliers de sols non-uniformes est décrite dans le texte, ainsi que l'investigation d'une surface de glissement complexe (plan de glissement non-circulaire). Ces deux enquêtes utilisent la méthode des tranches, et dans ce texte on explique la théorie de cette méthode. Une application pratique de la méthode des tranches est incluse dans le texte choisi no. 5.

Le troisième texte, ***Chart Solutions for Analysis of Earth Slopes*** (Tables de calcul pour l'analyse de talus en terre) a été publié dans l'Highway Research Record 345 (Highway Research Board, 1971). On y présente des solutions au problème de la stabilité des talus à l'aide de tables de calcul, et on met l'emphase sur l'application pratique de ces solutions plutôt que sur leurs dérivations. Dans le texte précédent on a décrit le développement des figures utilisées dans la solution dite de Taylor présentée dans ce texte. On inclut aussi dans ce texte (a) la solution de Bishop et Morgenstern, basée sur une adaptation suédoise de la méthode des tran-

This text provides a sampling of the charts developed for the solutions of a number of types of slope stability analyses. For more complete sets of charts for any of these solutions, the reader is referred to the references accompanying this text.

The fourth text, *Slope Design Guide*, is excerpted from *Transportation Engineering Handbook* (Region 6, U.S. Foreign Service, 1973). It was prepared for engineers and technicians who are required to design roads but are not skilled soils engineers or engineering geologists. It represents an attempt to reflect soils-engineering principles during the routine design of highway cut-and-fill slopes and, as such, requires many assumptions and simplifications. The procedures are not intended to replace the types of investigations and analyses described in the previous two texts or in the next text. Rather, they are meant to be an aid when more detailed investigations are not practical because of factors such as cost, value of the road involved, manpower, and available skill levels.

This guide is based on soil properties that are identified by the Unified classification system. (See Compendium 6, Selected Text 1, for comparisons of various soil classification systems.) It is usable without conducting laboratory soils

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es una solución más generalizada de la adaptación de Bishop del método sueco de tajadas; (d) la Solución de Hunter, que toma en consideración fluctuaciones del nivel freático; y (e) la Solución de Hunter y Schuster, que es un caso especial de la Solución de Hunter.

Este texto presenta un muestreo de los diagramas desarrollados para las soluciones de varios tipos de análisis de estabilidad de taludes. Si el lector desea un juego de diagramas más completo para cualquiera de estas soluciones, deberá referirse a las referencias incluídos al final de este texto.

El cuarto texto, *Slope Design Guide* (Guía para el diseño de taludes), fue extraído de *Transportation Engineering Handbook* (Manual para la ingeniería de transporte, Region 6, U.S. Forest Service, 1973). Fue preparado para los ingenieros y técnicos que son encargados del diseño de caminos, pero que no son especialistas en la ingeniería de suelos ni en la geología ingenieril. Trata de utilizar los principios de la ingeniería de suelos durante el diseño rutinario de taludes de corte y de relleno y, por lo tanto, requiere muchas suposiciones y simplificaciones. Estos procesos no intentan reemplazar los tipos de investigación y análisis descritos en los dos textos previos ni el texto siguiente. Se han diseñado para ayudar cuando investigaciones más detalladas no serían factibles por razón de factores como costo, valor del camino involucrado, mano de obra, y los niveles de habilidad disponibles.

La guía se basa en propiedades de suelo que se identifican por el sistema de clasificación unificada de suelos. (Veáse Compendio 6, Texto Seleccionado 1, para comparaciones entre varios sistemas de clasificación de suelos.) Se

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ches, (b) la solution de Morgenstern qui peut être utilisée pour des remblais routiers devant servir de digue à l'occasion, (c) la solution de Spencer qui est une solution généralisée de l'adaptation de Bishop de la méthode suédoise des tranches, (d) la solution de Hunter qui prend en compte les changements de niveau de la nappe phréatique, et (e) la solution de Hunter et Schuster qui s'applique à un cas spécial de la solution de Hunter.

Dans ce texte on trouvera un aperçu des tables de calcul développées pour effectuer plusieurs sortes d'analyse de stabilité de talus. Pour une série plus complète de tables de calcul pour chacune de ces solutions, le lecteur est invité à lire les références qui suivent ce texte.

Le quatrième texte, *Slope Design Guide* (Manuel de dimensionnement des talus), est extrait de *Transportation Engineering Handbook* (Region 6, U.S. Forest Service, 1973). Ce livre a été écrit à l'intention des ingénieurs et techniciens qui doivent concevoir et dimensionner des routes, mais ne sont ni specialistes de la technique des sols, ni géologues. On essaie de traiter des principes de la technique des sols pour le dimensionnement normal des talus routiers de remblai ou de déblai, et de ce fait, le texte assume et simplifie beaucoup de choses. Ces procédés ne devraient pas remplacer les investigations et les analyses décrites dans les deux textes antérieurs et dans celui qui va suivre. Nous les avons incluses pour les cas où il n'est pas possible de faire des analyses plus approfondies, à cause de considérations économiques ou de main d'oeuvre, de niveau d'expertise disponible, ou d'importance de la route en question.

Ce manuel est basé sur les caractéristiques des sols selon le système de la Classification Unifiée (voir le recueil no. 6, texte choisi no. 1, pour la comparaison de différents systèmes de classification des sols). On peut l'employer sans shear strength tests because it is based in part on soil identification as described in Compendium 2, Selected Text 6, and the simplified tests described in Compendium 7, Selected Text 3. The guide was developed from typical soil strength values by using chart solutions for slope stability, studies that use the conventional method of slices, published empirical relations, and the authors' experiences.

The following data were used to develop this guide:

1. The effect of seepage in coarse-grained materials was determined by using one-half the angle of internal friction ($\phi/2$) as the effective angle of internal friction for the high ground-water condition.

2. The angles of internal friction used for the development of the maximum slope ratios in ***Table II: Sands and Gravels with Nonplastic Fines*** (page 90) are as follows:

Soil Number	*Soil Type*	*Angle of Internal Friction (degrees)*	
		Loose	*Dense*
1	Sandy gravels	34	50
2	Well-graded sands, angular grains	32	45
3	Silty gravels and sands, uniform sands	27	34

3. The soil strength values used for the development of ***Charts I and II: Sands and Gravels with Plastic Fines*** are as follows:

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porque se basa en parte en la identificación de suelos, como se describe en el Compendio 2. Texto Seleccionado 4; identificación de rocas, como se describe en el Compendio 2. Texto Seleccionado 6; y las pruebas simplificadas descritas en el Compendio 7, Texto Seleccionado 3. La guía se desarrolló de valores típicos de resistencia del suelo utilizando soluciones gráficas para la estabilidad de taludes, estudios en que se utiliza el método convencional de tajadas, relaciones empíricas publicadas, y las experiencias de los autores.

Se utilizaron los siguientes datos para desarrollar esta guía:

1. El efecto de filtración en los materiales de granulación gruesa fue determinado utilizando la mitad del ángulo de fricción interna ($\phi/2$) como el ángulo efectivo de fricción interna para la condición de agua subterránea cerca de la superficie.

2. Los ángulos de fricción interna utilizados para el desarrollo de las razones máximas de talud en la ***Tabla II: Arenas y Gravas con Finos No Plásticos*** (página 90) son como sigue:

No (grados) de suelo	*Tipo de suelo*	*Angulo de fricción interna*	
		Suelto	*Denso*
1	gravas arenosas	34	50
2	arenas bien graduadas, granos angulares	32	45
3	gravas y arenas fangosas, arenas uniformes	27	34

3. Los Valores de Resistencia del Suelo que se utilizaron en el desarrollo de los ***Diagramas I y II: Arenas y Gravas con Finos Plásticos*** son como sigue:

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puede utilizar sin realizar pruebas de laboratorio de resistencia de suelos al esfuerzo cortante faire d'essais en laboratoire pour déterminer la résistance au cisaillement des sols, car il est basé en partie sur l'identification des sols telle qu'elle est décrite dans le recueil no. 2, texte choisi no. 4—***Identification des roches***—décrite dans le texte no. 6 du même recueil; et les essais simplifiés décrits dans le texte no. 3 du recueil no. 7. Le manuel a été développé à partir de valeurs typiques de résistance des sols en utilisant des tables de calcul pour la stabilité des talus, d'études qui utilisent la méthode conventionnelle des tranches, de rapports empiriques qui ont été publiés, et de l'expérience personnelle des auteurs.

On s'est servi des données ci-dessous pour développer ce guide:

1. L'influence de la percolation sur les matériaux granuleux grossiers a été calculée en utilisant la moitié de l'angle de frottement interne ($\phi/2$) comme angle de frottement interne réel, quand le niveau de l'eau phréatique est élevé.

2. Les angles de frottement interne utilisés pour calculer les coefficients maximaux des ta-

Soil Number	*Soil Type*	*Angle of Internal Friction (degrees)*	*Cohesion (lb/ft^2)*
1	(See page 93	20	1000
2	for description	15	750
3	of these soils.)	13	500
4		15	250
5		10	250

4. The soil strength values used for the development of *Charts III and IV: Fine Grained Soils* are as follows:

Soil Number	*Soil Type*	*Angle of Internal Friction (degrees)*	*Cohesion (lb/ft^2)*
1	(See pages 96-97	0	3000
2	for description	0	1500
3	of these soils.)	0	750
4		0	400
5		0	200

It must be emphasized that this guide must not be followed indiscriminately as a precise answer to all situations. It must be used in connection with local experience to arrive at reasonable values for slope ratios.

The fifth text, *Determining Corrective Action for Highway Landslide Problems*, taken from *Highway Research Board Bulletin 49* (Highway Research Board, 1955), presents the basic fundamentals of landslide analyses and classifies the corrective measures commonly used in controlling or avoiding highway landslide problems. It describes the preliminary analysis of a landslide, the detailed field study of the landslide area, and a stability analysis of an actual landslide in silty-clay soil overlying bedrock.

The stability analysis described is a composite of numerous methods that have appeared in the

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No de suelo	*Tipo de suelo*	*Angulo de fricción interna (grados)*	*Cohesión ($libras/pié^2$)*
1	véase la página 93	20	1000
2	para la descripción	15	750
3	de estos suelos	13	500
4		15	250
5		10	250

4. Los Valores de Resistencia del Suelo utilizados en el desarrollo de los *Diagramas III y IV: Suelos de Grano Fino* son como sigue:

No de suelo	*Tipo de suelo*	*Angulo de fricción interna (grados)*	*Cohesión ($libras/pié^2$)*
1	véase las	0	3000
2	páginas 96-97	0	1500
3	para la descripción	0	750
4	de estos suelos	0	400
5		0	200

Deberá subrayarse que esta guía no deberá seguirse indistintamente como respuesta precisa para toda situación. Deberá utilizarse en conexión con la experiencia local para llegar a

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lus de la table II: Sables et graviers avec fines non plastiques p. 90 sont les suivants:

N° du sol	*Type de sol*	*Angle de frottement interne (en degrés)*	
		non tassé	*dense*
1	graviers sableux	34	50
2	sable à bonne granulométrie, grains angulaires	32	45
3	graviers limoneux et sables, sable uniforme	27	34

3. Les valeurs de résistance des sols utilisées pour le développement des tables I et II: Sables et graviers avec fines plastiques sont les suivantes:

No. du sol	*Type de sol*	*Angle de frottement interne (en degrés)*	*Cohésion ($livres/pied^2$)*
1	voir page 93	20	1000
2	pour la	15	750
3	description	13	500
4		15	250
5		10	250

4. Les valeurs de résistance des sols utilisées dans les tables III et IV: Sols à grains fins sont les suivantes:

No. du sol	*Type de sol*	*Angle de frottement interne (en degrés)*	*Cohésion ($livres/pied^2$)*
1	voir pages 96-97	0	3000
2	pour la description	0	1500
3	de ces sols	0	750
4		0	400
5		0	200

Remarquons de nouveau que ce guide ne doit pas être utilisé sans discrimination, et comme ayant une réponse précise pour toutes sortes de situations. On doit l'utiliser en conjonction avec l'expérience locale, pour arriver à des valeurs raissonables de pentes de talus.

Le cinquième texte, *Determining Corrective Action for Highway Landslide Problems* (Détermination des mesures correctives pour résoudre les problèmes de glissements de terrain en literature and is recommended for use in all landslides involving unconsolidated material. It involves the determination of possible slip surfaces and their investigation by graphical integration (i.e., the summation of tangential and normal forces of a set of incremental areas measured on a scaled section of the slide). In the appendixes that accompany the paper, the actual stability analysis is demonstrated and then modified for evaluation of the hydrostatic pressure due to the presence of ground water. Additional computations from the same example show the technique for computing the size of a rock buttress near the toe of the slide to restrain the material. The computations used to evaluate the location and number of piles needed to stabilize the slide are also noted. Further computations are shown to evaluate the improved stability introduced by the installation of a drainage system to lower the ground-water table. The value of replacing the top of the slide (where the roadway is located) with lightweight fill is demonstrated by further calculations, as is the lowering of the road profile in the same location.

The various methods for solving landslide problems by (a) removal, (b) control, or (c) direct rebalance of the ratio between resistance and force are ranked in order of cost within each category. The following factors are noted for each of the various methods in each category: (a) description, (b) principle involved, (c) best application, (d) disadvantages, (e) method of analysis, and (f) principal items in the cost estimate.

The text concludes that, for a given highway-landslide problem, there are numerous solutions that can be satisfactorily applied, and the problem can be reduced to a problem in economics.

The sixth text, excerpted from ***Handbook on Landslide Analysis and Correction*** (Central Road

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valores razonables para razones de taludes.

El quinto texto, ***Determining Corrective Action for Highway Landslide Problems*** (Determinación de la operación correctiva para problemas de deslizamiento vial), de ***Highway Research Board Bulletin 49*** (Boletín 49 del Consejo de Investigación Vial, Highway Research Board, 1955), presenta los principios básicos del análisis de deslizamientos y clasifica las medidas correctivas que comúnmente se utilizan para controlar o evitar problemas de deslizamiento de carreteras. Describe el análisis preliminar de un deslizamiento, la investigación detallada de campo del área del deslizamiento, y un análisis de estabilidad de un deslizamiento verdadero en suelo arcilloso sedimentoso sobre roca basal.

El análisis de estabilidad que se describe es una mezcla de numerosos métodos que han aparecido en la literatura y se recomienda para cualquier deslizamiento que involucre material no consolidado. Incluye la determinación de posibles superficies de deslizamiento y su investigación por medio de la integración gráfica (es decir, la suma de fuerzas tangenciales y normales de un grupo de áreas incrementales medidas sobre una sección graduada del deslizamiento). En los apéndices que se incluyen con el informe, el análisis de estabilidad se demuestra y luego se modifica para la evaluación de la presión hidrostática debida a agua subterránea. Cálculos adicionales de la misma muestra demuestran la técnica para computar el tamaño de un contrafuerte de roca cerca del pie del deslizamiento para contener el material. También se anotan las computaciones que se

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construction routière) tiré du Highway Research Board Bulletin 49 (Highway Research Board, 1955), présente les principes fondamentaux de l'analyse des glissements de terrain, et classe les mesures correctives habituellement prises pour éviter ou contrôler les problèmes de glissements de terrain en construction routière. On y décrit l'analyse préliminaire d'un glissement de terrain, l'étude détaillée, sur le terrain, de ce glissement, et une analyse de stabilité d'un glissement de terrain actuel, dans de l'argile limoneuse posée sur une fondation rocheuse.

L'analyse de stabilité décrite ici est un composite de nombreuses méthodes publiées dans la litérature technique, et son emploi est recommandé pour tout glissement impliquant un matériau non-consolidé. On détermine les surfaces de glissement potentielles et leur investigation, en utilisant l'intégration graphique (i.e., l'addition de forces tangentielles et normales d'un groupe de surfaces différentielles mesurées sur une section divisée du glissement). Dans les annexes on fait la démonstration de l'analyse de stabilité, et ensuite on la modifie pour évaluer la pression hydrostatique dûe à la présence d'eau phréatique. D'autres calculs, tirés du même exemple, montrent comment évaluer la taille d'un contrefort en roc au pied du talus, pour retenir le matériau. Les calculs pour déterminer l'emplacement et le nombre de pieux

Research Institute, New Delhi, India, 1966), was written as a compilation of usable information condensed into a single volume for the practicing engineer who cannot invest the time to review the maze of engineering literature pertaining to landslides. Chapters 1 and 2 are not included here because the information is already presented, in updated form, in previous texts in Compendium 13. This handbook is not intended to eliminate the need for an expert or a specialist in the solution of all landslide problems.

The text covers slope design in bedrock cuts, ditch design in rockfall areas and the location for rock fences. It describes the characteristic features of landslides peculiar to different soil types and the field and laboratory investigations of landslides. It lists the techniques of prevention and correction of landslides, which is an expansion of the list provided in the previous text.

It concludes with the basic rules of analysis for prevention and correction of landslides that include (a) rules relating to the location of new lines of transportation in hills from the viewpoint of landslide prevention and (b) rules relating to field investigation of actual landslides with a view to planning control and corrective measures. The listing of the above rules contains much practical advice and also refers the reader

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utilizan para evaluar la ubicación y cantidad de pilotes que se necesitan para estabilizar el deslizamiento. Se indican más computaciones para evaluar la estabilidad mejorada producida por la instalación de un sistema de drenaje para el abatimiento del nivel freático. Asimismo hay cálculos que demuestran el valor del reemplazo de la parte superior del deslizamiento (donde se ubica el camino) con relleno de peso liviano, y también el rebajamiento del perfil del camino en la misma ubicación.

Los diversos métodos para resolver problemas de deslizamiento por (a) remoción, (b) control, o (c) un rebalanceo directo de la razón entre resistencia y fuerza han sido colocados en orden de costo dentro de cada categoría. Se han notado los siguientes factores para cada uno de los varios métodos en cada categoría: (a) descripción, (b) el principio involucrado, (c) la mejor aplicación, (d) desventajas, (e) método de análisis, y (f) partidas principales en el cálculo de costos.

El texto concluye que, para dado problema de deslizamiento vial, hay numerosas soluciones que pueden aplicarse satisfactoriamente, y que el problema puede reducirse a uno de costo.

El sexto texto, extraído de ***Handbook on Landslide Analysis and Correction*** (Manual sobre el análisis y corrección de deslizamientos, Central Road Research Institute, New Delhi, India, 1966), fue escrito como una compilación de información utilizable, condensada en un solo volumen, para el ingeniero en ejercicio que no tiene el tiempo necesario para repasar la cantidad de literatura ingenieril que se concierne con deslizamientos. Los Capítulos 1 y 2 no han sido incluídos porque la información ya ha sido presentada, en forma actualizada, en textos pre-

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nécessaires pour stabiliser le glissement sont aussi indiqués. On donne d'autres méthodes de calcul pour évaluer l'amélioration de la stabilité apportée par l'installation d'un système de drainage qui abaisse le niveau de la nappe phréatique. On démontre avec d'autres calculs l'avantage de remplacer la partie supérieure du glissement (où la route est située) par un matériau lèger, et celui de rabaisser le profil de la route au même endroit.

Les différentes solutions aux problèmes de glissements de terrain par (a) suppression, (b) contrôle, ou (c) en ré-équilibrant directement le rapport entre la résistance et la force sont rangées d'après le coût de chaque catégorie. Les facteurs suivants sont notés pour chaque méthode dans chaque catégorie: (a) description, (b) le principe en question, (c) la meilleure application, (d) les désavantages, (e) la méthode d'analyse, et (f) les points principaux de l'estimation des coûts.

En conclusion, il est décidé qu'a un problème donné de glissement de terrain routier, on peut trouver de nombreuses solutions qui peuvent être appliquées avec succés, et qu'en fin de compte, ce problème peut être réduit à un problème économique.

Le sixième texte, extrait de *Handbook on Landslide Analysis and Correction* (Manuel d'analyse et correction de glissements de terrain) publié par le Central Road Research Institute, New Delhi, India en 1966, est une compilation d'information utile, résumée en un seul volume, à l'intention de l'ingénieur sur le chantier, qui n'a pas le temps de passer en revue le dédale de littérature technique sur les glissements de terrain. Nous avons omis les chapitres 1 et 2, car les informations qu'ils contiennent sont déjà présentée, remises à jour, dans les textes précédents de ce recueil. Ce manuel n'est pas écrit avec l'intention d'éliminer le besoin d'un expert ou d'un spécialiste pour to the various previous sections of the text (or by inference to their substituted previous compendium texts) for details of specific problems or solutions.

The seventh text is excerpted from *Construction of Embankments (NCHRP Synthesis of Highway Practice No. 8*, Highway Research Board, 1971). It indicates that the strength of an embankment built with current standard-design slopes is not critical if proper materials and compaction are used. Selected Texts 2 and 4 of this compendium indicate that standard-design fill slopes of 6 to 1 or 4 to 1 far exceed the stability requirements of reasonable fill material. This text attributes most embankment failures to (a) soft foundation soils, (b) sidehill locations, (c) cut-fill transitions, and (d) ground-water problems.

Soft foundation soils (peats, marls, and organic and inorganic silts and clays) may be removed or consolidated. Sidehill fills increase the tendency of unstable foundation material to slide and disrupt the natural movement of surface water and ground water. Benching to key the embankment to a firm foundation and special drainage provisions may overcome these problems. Cut-fill transitions basically are transverse sidehill locations and may also require benching and special drainage. In order to maintain a uniform subgrade, the bench must extend far enough into the cut zone to remove all unstable soil from the subgrade zone. Ground water may be controlled by use of previous blankets or some type of drain-pipe system or by raising the embankment in flat terrain.

The design of highway fills generally consists

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vios del Compendio 13. Este manual no tiene el propósito de eliminar la necesidad de consultar con un experto o especialista sobre la solución de todos los problemas de deslizamientos.

El texto incluye el diseño de taludes en cortes de roca basal, el diseño de zanjas en áreas de desprendimientos de rocas y la ubicación de cercas guardarocas. Describe los elementos característicos de deslizamientos propios a distintos tipos de suelo y las investigaciones de campo y de laboratorio de deslizamientos. Nombra las técnicas de prevención y corrección de deslizamientos, como ampliación de la lista proveída en el texto previo.

Concluye con las reglas básicas de análisis para la prevención y corrección de deslizamientos que incluyen (a) las reglas que se relacionan con la ubicación de nuevas líneas de transporte en terreno accidentado desde el punto de vista de prevención de deslizamientos y (b) las reglas que se relacionan con la investigación de campo de deslizamientos existentes con vista al desarrollo de medidas de control y corrección. La lista de dichas reglas contiene muchos consejos útiles, y también indica para el lector las diversas secciones previas del texto (o por inferencia los textos previos que las substituyen) que describen con más detalle los problemas o soluciones específicos.

El séptimo texto fue extraído de *Construction of Embankments* del *NCHRP Synthesis of Highway Practice No. 8* (Construcción de terraplenes, Síntesis NCHRP de la práctica vial Nº 8, Highway Research Board, 1971). Indica que la resistencia de un terraplén construído según normas corrientes de diseño de pendientes no es crítica si se utilizan materiales y compactación correctos. Los Textos Seleccionados 2 y 4 de este compendio indican que las pendientes de relleno (construídas según las normas de diseño) de 6 a 1 ó 4 a 1 ampliamente exceden los requisitos de estabilidad de material de relleno

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résoudre tous les problèmes de glissements de terrain.

On couvre le dimensionnement des pentes de déblai rocheux, des fossés dans les endroits propices aux chûtes de pierres, et l'emplacement de barrières de protection contre les éboulements rocheux. On décrit les éléments caractéristiques de glissements particuliers aux différents types de sol, et les investigations en laboratoire et sur le chantier. On énumère les techniques de prévention et de correction des glissements de terrain. Cette liste est une expansion de celle donnée dans le texte précédent.

A la fin du texte on donne les règles de base des analyses pour la prévention et la correction des glissements de terrain: (a) règles sur l'emplacement de nouvelles lignes de transport en région montagneuse, du point de vue de la prévention des glissements de terrain, (b) règles pour l'investigation, sur le chantier, de glissements de terrain, au point de vue des mesures de contrôle et de correction de ceux-ci. Cette liste contient un grand nombre de conseils pratiques, et renvoit le lecteur aux différentes sections précédentes du texte (ou par déduction aux textes de ce recueil que nous leur avons substitué) pour les détails de problèmes ou de solutions spécifiques.

Le septième texte est extrait de *Construction of Embankments, NCHRP Synthesis of Highway Practice No. 8* (Construction de remof establishing the height and the side slopes of the embankment and of specifying criteria for placement of the fill. Strict adherence to balanced earthwork design can lead to serious construction and maintenance problems by encouraging the use of poor-quality soils from cut sections and the use of locations with poor foundation conditions. The text indicates that in relatively flat terrain many engineers now prefer to ignore balanced earthwork concepts and to construct continuous low embankments. The design load used to evaluate the stability and the deformation of an embankment is the weight of the overlying embankment and pavement materials. Except for the upper few feet, embankment stability is not seriously affected by traffic loads.

The eighth text is a paper entitled ***Locating Ground Water for Design of Subsurface Drainage in Roadways and Embankments*** (45th Annual Tennessee Highway Conference, University of Tennessee, 1963). It discusses the problem of failures of embankments built on sloping ground. As previously noted in Selected Texts 6 and 7, embankments frequently fail (slip out) due to blockage of natural surface or underground water courses, especially embankments built on layered sedimentary deposits.

Ground water frequently appears at the ground surface as permanent or intermittent (wet-weather) springs. These springs are of three types: (a) fissure springs, (b) tubular springs, or (c) seepage springs. Identification of these various types of springs and the location

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razonable. Este texto atribuye muchas de las fallas de terraplén a (a) suelos de fundación blandos, (b) ubicaciones sobre la ladera de una colina, (c) transiciones de corte-relleno, y (c) problemas de agua subterránea.

Los suelos blandos en la fundación (turbas, margas, y limos y arcillas orgánicos e inorgánicos) pueden ser removidos o consolidados. Los rellenos sobre laderas de colinas aumentan la tendencia de materiales de fundación inestables hacia el deslizamiento y la interrupción consiguiente del movimiento natural de agua de superficie y agua subterránea. El banqueo del terraplén para calzarlo a una fundación firme, y el establecimiento de provisiones especiales de drenaje podrían ayudar a superar estos problemas. Básicamente las transiciones de corte-relleno son ubicaciones transversas sobre laderas de colinas y también pueden requerir el

banqueo y drenaje especial. Para mantener una subrasante uniforme el banqueo deberá penetrar la zona de corte lo suficiente para quitar todo suelo inestable de la zona de subrasante. El agua freática puede controlarse con el uso de delantales permeables de drenaje o algún tipo de sistema de tubos de drenaje, o elevando el terraplén en terreno llano.

El diseño de terraplenes viales consiste generalmente en el establecimiento de la altura y las pendientes laterales del terraplén y la especificación de los criterios para la colocación del relleno. Si se adhiere estrictamente a la teoría de equivalencias en el movimiento de tierras, pueden surgir graves problemas de construcción y conservación ya que se recomienda el uso de suelos de mala calidad de secciones de corte y el uso de ubicaciones con condiciones pobres de fundamento. El texto indica que hoy en día

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blais — NCHRP synthèse de pratique routière no. 8). On y indique que la résistance d'un remblai construit selon les normes de dimensionnement actuellement en vigueur, ne sera pas critique si on utilise des matériaux convenables, compactés correctement. Les textes 2 et 4 de ce recueil indiquent que les normes de pentes de remblai de 6 pour 1 ou de 4 pour 1 excèdent largement les conditions requises de stabilité d'un matériau de remblai convenable. On attribue la plupart des ruptures de remblais à (a) un sol de fondation mou, (b) un remblai situé sur un versant naturel, (c) des raccordements remblai-déblai et (d) des problèmes d'eau phréatique.

Les sols de fondation mous (tourbes, marnes, limons et argiles organiques ou non) peuvent être consolidés ou enlevés. Les remblais sur les versants naturels augmentent la tendance au glissement des matériaux de fondation instables, et dérangent l'écoulement naturel de l'eau de surface et de l'eau phréatique. On peut construire des redans ou gradins pour ancrer le remblai sur un fondation ferme, et construire un dispositif de drainage spécial pour essayer de surmonter ces problèmes. Les raccordements remblai-déblai sont fondamentalement des emplacements transversaux sur le versant, et peuvent aussi demander des redans et un dispositif drainant spécial. Pour conserver un sous-sol uniforme il faut étendre le redan aussi loin que nécessaire dans la zone de déblai, pour que tout sol instable soit enlevé de la zone de sous sol. On peut contrôler l'eau phréatique en installant des masques drainants, ou quelque sorte de tuyau de drainage, ou encore en élevant le remof their underground supply passages when the existing ground has been stripped for embankment placement will provide the low-volume road engineer with a very good indication of areas of possible fill failures. If the embankment is constructed of impervious materials or if placement methods result in a very dense embankment, drainage must be provided for all disturbed water passages or embankment failures will occur.

If an embankment fails during or after construction due to excessive soil moisture, the location of sources of water that contribute to the failure is much more difficult. Unless the flow is diverted, however, the embankment will fail again after being repaired. This paper discusses the location of ground water (a) before any construction activity has taken place, (b) after pioneer roads and slope benches in fill areas have been constructed, and (c) during or after construction.

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en áreas de terreno llano muchos ingenieros prefieren dejar de lado los conceptos de dicha teoría y construir terraplenes bajos contínuos. La carga de diseño utilizada para evaluar la estabilidad y la deformación de un terraplén es el peso del terraplén y los materiales de pavimentación que lo cubren. La única parte del terraplén gravemente afectada en su estabilidad por el tránsito, es los primeros piés de la parte superior.

El octavo texto es un informe titulado ***Locating Ground Water for Design of Subsurface Drainage in Roadways and Embankments*** (Localización de agua freática para el diseño de drenaje subálveo en caminos y terraplenes, 45th Annual Tennessee Highway Conference, University of Tennessee, 1963). Habla sobre el problema de las fallas de terraplenes construídos sobre depósitos sedimentarios en capas.

Es frecuente que el agua freática aparece en la superficie en forma de manantiales permanentes o intermitentes (de tiempo de lluvias). Estos manantiales son de tres tipos: (a) manantiales de grietas, (b) manantiales tubulares, o (c) manantiales de filtración. Si se identifican estos diversos tipos de manantiales y se ubican los pasajes subterráneos que los alimentan cuando se ha desbrozado el terreno circundante para la colocación del terraplén, el ingeniero de caminos de bajo volumen tendrá una idea bastante precisa de las posibles áreas de falla del terraplén. No se puede evitar una falla

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blai dans les terrains plats.

Le dimensionnement des remblais consiste généralement à établir la hauteur et les pentes du remblai, et à spécifier les critères de mise en oeuvre du remblai. On peut se créer de sérieux problèmes de construction et d'entretien, si l'on suit trop strictement les principes de l'équilibre des terrassements en encourageant l'utilisation de sols de mauvaise qualité provenant des déblais, et d'emplacements où la fondation est de mauvaise qualité. Le texte indique que dans un terrain relativement plat beaucoup d'ingénieurs routiers préférent ignorer le concept de l'équilibre des terrassements, et construire des remblais continus et pas très hauts. L'hypothèse de charge utilisée pour évaluer la stabilité et la déformation d'un remblai est égale au poids du remblai plus celui du revêtement routier. Sauf pour quelques pieds de la partie supérieure du remblai, les charges de la circulation n'ont pas d'éffet sérieux sur sa stabilité.

Le huitième texte est une communication intitulée ***Locating Ground Water for Design of Subsurface Drainage in Roadways and Embankments*** (Localisation de l'eau phréatique pour le dimensionnement d'un dispositif de drainage souterrain pour routes et remblais, 45th Annual Tennessee Highway Conference, University of Tennessee, 1963). On y discute le problème de la rupture des remblais construits sur des terrains déclives. Comme nous l'avons remarqué dans les textes choisis no. 6 et 7, il y a souvent rupture de remblai (glissement) quand l'écoulement naturel de l'eau de surface ou souterraine est bloqué, surtout si le remblai est bâti sur des couches de terrain sédimentaire.

L'eau souterraine apparait fréquemment à la surface du sol en tant que source permanente ou intermittente, par exemple seulement à la saison des pluies. Ces sources sont de trois sortes: (a) les sources dans les petites crevasses des rochers, (b) les sources tubulaires ainsi nommées car leur cours souterrain est de forme tubulaire et (c) les sources de percolation ou filtration. L'identification de ces différentes sortes de sources, et la localisation de leur cours souterrain quand le terrain naturel a été enlevé pour la construction de remblai, donnera à l'ingénieur de routes économiques, de précieuçes indications sur les zones de rupture potentielles. Si le remblai est construit de matériaux imperméables, ou si les méthodes de mise en oeuvre ré-

Bibliography

The selected texts are followed by a brief bibliography containing reference data and abstracts for 20 publications. The first eight describe the selected texts. The other 12 describe publications related to the selected texts. Although there are many articles, reports, and books that could be listed, it is not the purpose of this bibliography to contain all possible references related to the subject of this compendium. The bibliography contains only those publications from which a text has been selected or basic publications that would have been selected had there been no page limit for this compendium.

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en el terraplén si éste se construye de materiales impermeables, si el método de colocación produce un terraplén muy denso, o si no se proporciona el drenaje necesario para todas las vías de agua interrumpidas.

Si debido a mucha humedad, falla un terraplén durante o después de la construcción, se vuelve mucho más difícil localizar las fuentes del agua que contribuyen al problema. No obstante, si no se desvía el flujo, el terraplén fallará otra vez después de repararse. Este papel habla sobre la localización de agua freática: (a) antes de comenzar con las actividades de construcción, (b) después de que se hayan construído caminos precursores y bancos de pendiente en las áreas de relleno, y (c) durante o después de construcción.

Bibliografía

Al final de los textos seleccionados el lector encontrará una breve bibliografía que contiene los datos y abstractos de referencia para 20 publicaciones. Las primeras ocho referencias describen los textos seleccionados. Las otras 12 describen publicaciones relacionadas con los textos seleccionados. Aunque existen muchos artículos, informes, y libros que podrían nombrarse, no es el propósito de esta bibliografía mencionar todas las posibles referencias que se relacionen con el tema de este compendio. Contiene únicamente aquellas publicaciones de las cuales se ha seleccionado un texto y las publicaciones básicas que se habrían seleccionado si no hubiera un límite al número de páginas en este compendio.

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sultent en un remblai très dense, il est impératif d'installer un dispositif de drainage pour toutes les eaux dont le cours a été détourné si l'on veut éviter la rupture.

Si, pendant ou après la construction, on a une rupture de remblai causée par l'humidité excessive du sol, il est beaucoup plus difficile de localiser la source d'eau qui a contribué à cette rupture. Il y aura de nouveau rupture de ce remblai si l'écoulement n'est pas détourné avant la reconstruction. Dans cette communication, on discute comment localiser l'eau phréatique:
(a) avant de commencer la construction, (b) au stade exploratoire de la construction de la route et des redans dans les zones de remblai et (c) pendant ou après la construction.

Bibliographie

Les textes choisis sont suivis d'une courte bibliographie contenant les références et résumés de 20 publications. Les huit premiers décrivent les textes choisis. Les autres douze décrivent des textes apparentés au sujet des textes choisis. Bien qu'il existe beaucoup d'articles, rapports et livres que nous pourrions énumérer, l'objectif de cette bibliographie n'est pas d'inclure toute la littérature publié sur le sujet de ce recueil. Cette bibliographie contient seulement les publications dont nous avons extrait un texte, ou des publications de base que nous aurions aimé, mais n'avons pû inclure, pour des raisons évidentes de concision.

Project Description

The development of agriculture, the distribution of food, the provision of health services, and the access to information through educational services and other forms of communication in rural regions of developing countries all heavily depend on transport facilities. Although rail and water facilities may play important roles in certain areas, a dominant and universal need is for road systems that provide an assured and yet relatively inexpensive means for the movement of people and goods. The bulk of this need is for low-volume roads that generally carry only 5 to 10 vehicles a day and that seldom carry as many as 400 vehicles a day.

The planning, design, construction, and maintenance of low-volume roads for rural regions of developing countries can be greatly enhanced with respect to economics, quality, and performance by the use of low-volume road technology that is available in many parts of the world. Much of this technology has been produced during the developmental phases of what are now the more developed countries, and some is continually produced in both the less and the more developed countries. Some of the technology has been documented in papers, articles, and reports that have been written by experts in the field. But much of the technology is

Descripción del proyecto

En las regiones rurales de países en desarrollo, el desarrollo de la agricultura, la distribución de víveres, la provisión de servicios de sanidad, y el acceso a información por medio de servicios educacionales y otras formas de comunicación, dependen en gran parte de los medios de transporte. Aunque en ciertas áreas los medios de ferrocarril y agua desempeñan un papel importante, existe una necesidad universal y dominante de crear sistemas viales que provean un medio asegurado pero relativamente poco costoso para el movimiento de gente y mercancías. La mayor parte de esta necesidad se solucionaría con la construcción de caminos de bajo volúmen que generalmente moverían únicamente de 5 a 10 vehículos por día y que pocas veces moverían tanto como 400 vehículos por día.

El planeamiento, diseño, construcción y mantenimiento de caminos de bajo volúmen para regiones rurales de países en desarrollo pueden ser mejorados, con respecto al costo, calidad, y rendimiento, por el uso de la tecnología de caminos de bajo volúmen que se encuentra disponible en muchas partes del mundo. Mucha de esta tecnología ha sido producida durante las épocas de desarrollo de lo que ahora son los países más desarrollados, y alguna se produce contínuamente en estos países así como en los países menos desarrollados. Parte de la tecnología se ha documentado en disertaciones, artículos, e informes que han sido escritos por expertos en el campo. Pero mucha de la tecnología no está documentada y existe principalmente en la memoria de aquellos que han desa-

Description du projet

Dans les régions rurales des pays en voie de développement, l'exploitation agricole, la distribution des produits alimentaires, l'accès aux services médicaux, l'accès aux matériaux et aux marchandises, à l'information et aux autres services, dépendent en grande partie des moyens de transport. Bien que les transports par voie ferrée et par voie navigable jouent un rôle important dans certaines régions, un besoin dominant et universel éxiste d'un réseau routier qui puisse assurer avec certitude et d'une façon relativement bon marché, le déplacement des habitants, et le transport des marchandises. La plus grande partie de ce besoin peut être satisfaite par la construction de routes à faible capacité, capables d'accommoder un trafic de 5 a 10 véhicules par jour, ou plus rarement, jusqu'à 400 véhicules par jour.

L'utilisation des connaissances actuelles en technologie, qui sont accéssibles dans beauundocumented and exists mainly in the minds of those who have developed and applied the technology through necessity. In either case, existing knowledge about low-volume road technology is widely dispersed geographically, is quite varied in the language and the form of its existence, and is not readily available for application to the needs of developing countries.

In October 1977 the Transportation Research Board (TRB) began this 3-year special project under the sponsorship of the U.S. Agency for International Development (AID) to enhance rural transportation in developing countries by providing improved access to existing information on the planning, design, construction, and maintenance of low-volume roads. With advice and guidance from a project steering committee, TRB defines, produces, and transmits information products through a network of correspondents in developing countries. Broad goals for the ultimate impact of the project work are to promote effective use of existing information in the economic development of transportation infrastructure and thereby to enhance other aspects of rural development throughout the world.

In addition to the packaging and distribution of technical information, personal interactions with users are provided through field visits, con-

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rrollado y aplicado la tecnología por necesidad. En cualquier caso, los conocimientos en existencia sobre la tecnología de caminos de bajo volúmen están grandemente esparcidos geográficamente, varian bastante con respecto al idioma y su forma, y no se encuentran fácilmente disponibles para su aplicación a las necesidades de los países en desarrollo.

En octubre de 1977 el Transportation Research Board (TRB) comenzó este proyecto especial de tres años de duración bajo el patrocinio de la U.S. Agency for International Development (AID) para mejorar el transporte rural en los países en desarrollo acrecentando la disponibilidad de la información en existencia sobre el planeamiento, diseño, construcción, y mantenimiento de caminos de bajo volúmen. Con el consejo y dirección de un comité de iniciativas para el proyecto, el TRB define, produce, y transmite productos informativos a través de una red de corresponsales en países en desarrollo. Las metas generales para el impacto final del trabajo del proyecto son la promoción del uso efectivo de la información en existencia en el desarrollo económico de la infraestructura de transporte y de esta forma mejorar otros aspectos del desarrollo rural a través del mundo.

Además de la recolección y distribución de la

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coup de pays, peut faciliter l'étude des projets de construction, tracé et entretien, de routes à faible capacité dans les régions rurales des pays en voie de développement, surtout en ce qui concerne l'économie, la qualité, et la performance de ces routes. La majeure partie de cette technologie a été produite durant la phase de développement des pays que l'on appelle maintenant développés, et elle continue à être produite à la fois dans ces pays et dans les pays en voie de développement. Certains aspects de cette technologie ont été documentés dans des articles ou rapports écrits par des experts. Mais une grande partie des connaissances n'existe que dans l'esprit de ceux qui ont eu besoin de développer et appliquer cette technologie. De plus, dans ces deux cas, les écrits et connaissances sur la technologie des routes à faible capacité, sont dispersés géographiquement, sont écrits dans des langues différentes, et ne sont pas assez aisément accessibles pour être appliqués aux besoins des pays en voie de développement.

En octobre 1977, le Transportation Research Board (TRB) initia ce projet, d'une duré de 3 ans, sous le patronage de l'U.S. Agency for International Development (AID), pour améliorer le transport rural dans les pays en voie de développement, en rendant plus accessible la documentation existante sur la conception, le tracé, la construction, et l'entretien des routes à faible capacité. Avec le conseil, et sous la conduite d'un comité de direction, TRB définit, produit, et transmet cette documentation à l'aide d'un réseau de correspondants dans les pays en voie de développement. Nous espérons que le résultat final de ce projet sera de favoriser l'utilisation de cette documentation, pour aider au développement économique de l'infrastructure des transports, et de cette façon mettre en valeur d'autres aspects d'exploitation rurale à travers le monde.

ferences in the United States and abroad, and other forms of communication.

Steering Committee

The Steering Committee is composed of experts who have knowledge of the physical and social characteristics of developing countries, knowledge of the needs of developing countries for transportation, knowledge of existing transportation technology, and experience in its use.

Major functions of the Steering Committee are to assist in the definition of users and their needs, the definition of information products that match user needs, and the identification of informational and human resources for development of the information products. Through its membership the committee provides liaison with project-related activities and provides guidance for interactions with users. In general the Steering Committee gives overview advice and direction for all aspects of the project work.

The project staff has responsibility for the preparation and transmittal of information products, the development of a correspondence network throughout the user community, and interactions with users.

Information Products

Three types of information products are prepared: compendiums of documented information on relatively narrow topics, syntheses of knowledge and practice on somewhat broader

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información técnica, se provee acciones recíprocas personales con los usuarios por medio de visitas de campo, conferencias en los Estados Unidos de Norte América y en el extranjero, y otras formas de comunicación.

Comité de iniciativas

El comité de iniciativas se compone de expertos que tienen conocimiento de las características físicas y sociales de los países en desarrollo, conocimiento de las necesidades de transporte de los países en desarrollo, conocimiento de la tecnología de transporte en existencia, y experiencia en su uso.

Las funciones importantes del comité de iniciativas son las de ayudar en la definición de usuarios y sus necesidades, de productos informativos que se asemejan a las necesidades del usuario, y la identificación de recursos de conocimientos y humanos para el desarrollo de los productos informativos. A través de sus miembros el comité provee vínculos con actividades relacionadas con el proyecto y también una guía para la interacción con los usuarios. En general el comité de iniciativas proporciona consejos y dirección general para todos los aspectos del trabajo de proyecto.

El personal de proyecto es responsable de la preparación y transmisión de los productos informativos, el desarrollo de una red de corresponsales a través de la comunidad de usuarios, y la interacción con los usuarios.

Productos informativos

Se preparan tres tipos de productos informativos: los compendios de la información documentada sobre temas relativamente limitados, la síntesis del conocimiento y práctica sobre temas

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En plus de la dissémination de cette documentation technique, des visites, des conférences aux Etats Unis et à l'étranger, et d'autres formes de communication permettront une interaction constante avec les usagers.

Comité de direction

Le comité de direction est composé d'experts qui ont à la fois des connaissances sur les caractéristiques physiques et sociales des pays en voie de développement, sur leurs besoins au point de vue transports, sur la technologie actuelle des transports, et ont aussi de l'expérience quant à l'utilisation pratique de cette technologie.

Les fonctions majeures de ce comité sont d'abord d'aider à définir les usagers et leurs besoins, puis de définir leurs besoins en matière de documentation, et d'identifier les ressources documentaires et humaines nécessaires pour le développement de cette documentation. Par l'intermédiaire des ses membres, le comité pourvoit à la liaison entre les différentes fonctions relatives au projet, et dirige l'interaction avec les usagers. En général, le comité de direction conseille et dirige toutes les phases du projet.

Notre personnel est responsable de la préparation et de la dissémination des documents, du développement d'un réseau de correspondants pris dans la communauté d'usagers, et de l'interaction avec les usagers.

La documentation

Trois genres de documents sont preparés: des recueils dont le sujet est relativement limité, des subjects, and proceedings of low-volume road conferences that are totally or partially supported by the project. Compendiums are prepared by project staff at the rate of about 6 per year; consultants are employed to prepare syntheses at the rate of 2 per year. At least one conference proceedings will be published during the 3-year period. In summary, this project aims to produce and distribute between 20 and 30 publications that cover much of what is known about low-volume road technology.

Interactions With Users

A number of mechanisms are used to provide interactions between the project and the user community. Project news is published in each issue of *Transportation Research News.* Feedback forms are transmitted with the information products so that recipients have an opportunity to say how the products are beneficial and how they may be improved. Through semiannual visits to developing countries, the project staff acquires first-hand suggestions for the project work and can assist directly in specific technical problems. Additional opportunities for interaction with users arise through international and in-country conferences in which there is project participation. Finally, annual colloquiums are held for students from developing countries who are enrolled at U.S. universities.

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un poco más amplios, y los expedientes de conferencias de caminos de bajo volúmen que están totalmente o parcialmente amparados por el proyecto. El personal de proyecto prepara los compendios a razón de unos 6 por año; se utilizan consultores para preparar las síntesis a razón de 2 por año. Se publicará por lo menos un expediente de conferencia durante el período de tres años. En breve, este proyecto pretende producir y distribuir entre 20 y 30 publicaciones que cubren mucho de lo que se conoce de la tecnología de caminos de bajo volúmen.

Interacción con los usuarios

Se utilizan varios mecanismos para proveer las interacciones entre el proyecto y la comunidad de usuarios. Se publican las noticias del proyecto en cada edición de la *Transportation Research News.* Se transmiten, con los productos informativos, formularios de retroacción para que los recipientes tengan oportunidad de decir cómo benefician los productos y cómo pueden ser mejorados. A través de visitas semianuales a los países en desarrollo, el personal del proyecto adquiere directamente de fuentes originales sugerencias para el trabajo del proyecto y puede asistir directamente en problemas técnicos específicos. Surgen oportunidades adicionales para la interacción con los usuarios a través de conferencias internacionales y nacionales en donde participa el proyecto. Finalmente, se organizan diálogos con estudiantes de países en desarrollo que están inscriptos en universidades norteamericanas.

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synthèses de connaissances et de pratique sur des sujets beaucoup plus généraux, et finalement des comptes-rendus de conférences sur les routes à faible capacité, qui seront organisées complètement ou en partie par notre projet. Environ 6 recueils par an sont preparés par notre personnel. Deux synthèses par an sont écrites par des experts pris à l'extérieur. Les comptes-rendus d'au moins une conférence seront écrits dans une période de 3 ans. En résumé, l'objet de ce projet est de produire et disséminer entre 20 et 30 documents qui couvriront l'essentiel des connaissances sur la technologie des routes à faible capacité.

Interaction avec les usagers

Un certain nombre de mécanismes sont utilisés pour assurer l'interaction entre le personnel du projet et la communauté d'usagers. Un bulletin d'information est publié dans chaque numéro de *Transportation Research News.* Des formulaires sont joints aux documents, afin que les usagers aient l'opportunité de juger de la valeur de ces documents et de donner leur avis sur les moyens de les améliorer. Au cours de visites semi-annuelles dans les pays en voie de développement notre personnel obtient de première main des suggestions sur le bon fonctionnement du projet et peut aider à résoudre sur place certains problèmes techniques spécifiques. En outre, des conférences tenues soit aux Etats Unis, soit à l'étranger, sont l'occasion d'un échange d'idées entre notre personnel et les usagers. Finalement, des colloques annuels sont organisés pour les étudiants des pays en voie de développement qui étudient dans les universités américaines.

Selected Texts

This section of the compendium contains selected pages from each text that is listed in the table of contents. Rectangular frames are used to enclose pages that have been reproduced from the original publication. Some of the original pages have been reduced in size to fit inside the frames. No other changes have been made in the original material except for the insertion of occasional explanatory notes. Thus, any errors that existed in the selected text have been reproduced in the compendium itself.

Page numbers of the original text appear inside the frames. Page numbers for the compendium are outside the frames and appear in the middle left or middle right outside margins of the pages. Page numbers that are given in the table of contents and in the index refer to the compendium page numbers.

Each text begins with one or more pages of introductory material that was contained in the original publication. This material generally includes a title page, or a table of contents, or both. Asterisks that have been added to original tables of contents have the following meanings:

*Some pages (or parts of pages) in this part of the original document appear in the

Textos seleccionados

Esta sección del compendio contiene páginas seleccionadas de los textos catalogados en la tabla de materias. Se utilizan recuadros rectangulares para encerrar las páginas que han sido reproducidas de la publicación original. Algunas de las páginas originales han sido reducidas para entrar en los recuadros. No se han hecho ningunos otros cambios en el material original exceptuando algunas notas aclaradoras que de vez en cuando han sido agregadas. De esta forma, cualquier error que hubiera existido en el texto seleccionado ha sido reproducido en el compendio mismo.

Los números de página del texto original aparecen dentro de los recuadros. Los números de página para el compendio están fuera de los recuadros y aparecen en el centro del márgen izquierdo o derecho de cada página. Los números de página que se dan en el índice del compendio se refieren a los del compendio.

Cada texto comienza con una o más páginas de material de introducción que contenía la publicación original. Este material generalmente incluye una página título, un índice, o ambos. Los asteriscos que han sido agregados al índice original significan lo siguiente:

*Algunas páginas (o partes de página) en esta parte del documento original aparecen

Textes choisis

Cette partie du recueil contient les sections extraites des publications indiquées à la table des matières. Les pages du texte original qui sont reproduites, sont entourées d'un encadrement rectangulaire. Certaines pages ont dû être réduites pour pouvoir être placées dans l'encadrement. Le texte original n'a pas été changé sauf pour quelques explications qui ont été insérées. Donc, si le texte original contient des erreurs, elles sont reproduites dans le recueil.

La pagination originale apparaît à l'intérieur de l'encadrement. La pagination du recueil est à l'extérieur de l'encadrement, soit à droite, soit à gauche de la marge extérieure des pages, et est celle qui est citée dans la table des matières et dans l'index du recueil.

Chaque texte commence par une ou plusieurs pages d'introduction qui étaient incluses dans le texte original. Ces pages sont généralement le titre, ou la table des matières, ou les deux. Des astériques ont été ajoutés à la table des matières d'origine, pour les raisons suivantes :

*Certaines pages, ou portions des pages, dans cet extrait du document original sont selected text, but other pages (or parts of pages) in this part of the original publication have been omitted.

**All pages in this part of the original document appear in the selected text.

The selected texts therefore include only those parts of the original documents that are preceded by asterisks in the tables of contents of the respective publications.

Broken lines across any page of selected text indicate those places where original text has been omitted. In a number of places, the selected text contains explanatory notes that have been inserted by the project staff. Such notes are set off within dashed-line boxes and begin with the word NOTE.

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Les lignes brisées sur les pages des textes choisis indiquent les endroits où le texte original a été omis. A certains endroits, les textes choisis contiennent des explications qui ont été insérées par notre personnel. Ces explications sont entourées d'un encadrement en pointillé, et commencent toujours par le mot NOTE.

LANDSLIDES
Analysis and Control

Special Report 176

Robert L. Schuster
Raymond J. Krizek
editors

Transportation Research Board
Commission on Sociotechnical Systems
National Research Council

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Contents

Chapter 2

Slope Movement Types and Processes

David J. Varnes

This chapter reviews a fairly complete range of slope-movement processes and identifies and classifies them according to features that are also to some degree relevant to their recognition, avoidance, control, or correction. Although the classification of landslides presented in Special Report 29 (*2.182*) has been well received by the profession, some deficiencies have become apparent since that report was published in 1958; in particular, more than two dozen partial or complete classifications have appeared in various languages, and many new data on slope processes have been published.

One obvious change is the use of the term slope movements, rather than landslides, in the title of this chapter and in the classification chart. The term landslide is widely used and, no doubt, will continue to be used as an all-inclusive term for almost all varieties of slope movements, including some that involve little or no true sliding. Nevertheless, improvements in technical communication require a deliberate and sustained effort to increase the precision associated with the meaning of words, and therefore the term slide will not be used to refer to movements that do not include sliding. However, there seems to be no single simple term that embraces the range of processes discussed here. Geomorphologists will see that this discussion comprises what they refer to as mass wasting or mass movements, except for subsidence or other forms of ground sinking.

The classification described in Special Report 29 is here extended to include extremely slow distributed movements of both rock and soil; those movements are designated in many classifications as creep. The classification also includes the increasingly recognized overturning or toppling failures and spreading movements. More attention is paid to features associated with movements due to freezing and thawing, although avalanches composed mostly of snow and ice are, as before, excluded.

Slope movements may be classified in many ways, each having some usefulness in emphasizing features pertinent to recognition, avoidance, control, correction, or other purpose for the classification. Among the attributes that have been used as criteria for identification and classification are type of movement, kind of material, rate of movement, geometry of the area of failure and the resulting deposit, age, causes, degree of disruption of the displaced mass, relation or lack of relation of slide geometry to geologic structure, degree of development, geographic location of type examples, and state of activity.

The chief criteria used in the classification presented here are, as in 1958, type of movement primarily and type of material secondarily. Types of movement (defined below) are divided into five main groups: falls, topples, slides, spreads, and flows. A sixth group, complex slope movements, includes combinations of two or more of the other five types. Materials are divided into two classes: rock and engineering soil; soil is further divided into debris and earth. Some of the various combinations of movements and materials are shown by diagrams in Figure 2.1 (in pocket in back of book); an abbreviated version is shown in Figure 2.2. Of course, the type of both movement and

Figure 2.2. Abbreviated classification of slope movements. (Figure 2.1 in pocket in back of book gives complete classification with drawings and explanatory text.)

TYPE OF MOVEMENT			TYPE OF MATERIAL		
			BEDROCK	ENGINEERING SOILS	
				Predominantly coarse	Predominantly fine
FALLS			Rock fall	Debris fall	Earth fall
TOPPLES			Rock topple	Debris topple	Earth topple
SLIDES	ROTATIONAL	FEW UNITS	Rock slump	Debris slump	Earth slump
	TRANSLATIONAL	FEW UNITS	Rock block slide	Debris block slide	Earth block slide
		MANY UNITS	Rock slide	Debris slide	Earth slide
LATERAL SPREADS			Rock spread	Debris spread	Earth spread
FLOWS			Rock flow (deep creep)	Debris flow (soil creep)	Earth flow (soil creep)
COMPLEX			Combination of two or more principal types of movement		

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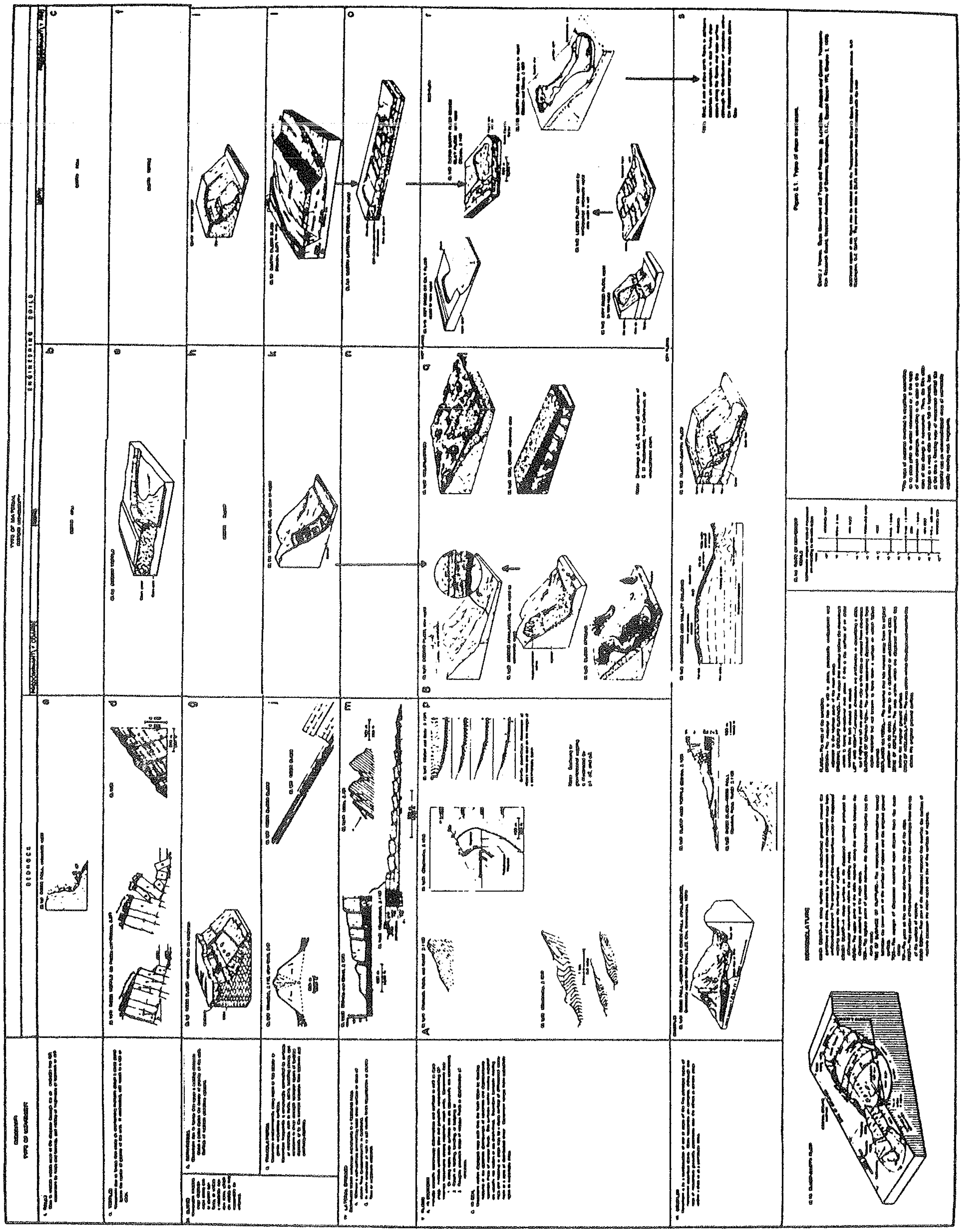


NOTE: Figure 2.1 (above) is included as a fold out in the back of this compendium.

materials may vary from place to place or from time to time, and nearly continuous gradation may exist in both; therefore, a rigid classification is neither practical nor desirable. Our debts to the earlier work of Sharpe (*2.146*) remain and are augmented by borrowings from many other sources, including, particularly, Skempton and Hutchinson (*2.154*), Nemčok, Pašek, and Rybář (*2.116*), de Freitas and Watters (*2.37*), Záruba and Mencl (*2.193*), and Zischinsky (*2.194*). Discussions with D. H. Radbruch-Hall of the U.S. Geological Survey have led to significant beneficial changes in both content and format of the presentation.

The classification presented here is concerned less with affixing short one- or two-word names to somewhat complicated slope processes and their deposits than with developing and attempting to make more precise a useful vocabulary of terms by which these processes and deposits may be described. For example, the word creep is particularly troublesome because it has been used long and widely, but with differing meanings, in both the material sciences, such as metallurgy, and in the earth sciences, such as geomorphology. As the terminology of physics and materials science becomes more and more applied to the behavior of soil and rock, it becomes necessary to ensure that the word creep conveys in each instance the concept intended by the author. Similarly, the word flow has been used in somewhat different senses by various authors to describe the behavior of earth materials. To clarify the meaning of the terms used here, verbal definitions and discussions are employed in conjunction with illustrations of both idealized and actual examples to build up descriptors of movement, material, morphology, and other attributes that may be required to characterize types of slope movements satisfactorily.

TERMS RELATING TO MOVEMENT

Kinds of Movement

Since all movement between bodies is only relative, a description of slope movements must necessarily give some attention to identifying the bodies that are in relative motion. For example, the word slide specifies relative motion between stable ground and moving ground in which the vectors of relative motion are parallel to the surface of separation or rupture; furthermore, the bodies remain in contact. The word flow, however, refers not to the motions of the moving mass relative to stable ground, but rather to the distribution and continuity of relative movements of particles within the moving mass itself.

Falls

In falls, a mass of any size is detached from a steep slope or cliff, along a surface on which little or no shear displacement takes place, and descends mostly through the air by free fall, leaping, bounding, or rolling. Movements are very rapid to extremely rapid (see rate of movement scale, Figure 2.1u) and may or may not be preceded by minor movements leading to progressive separation of the mass from its source.

Rock fall is a fall of newly detached mass from an area of bedrock. An example is shown in Figure 2.3. Debris fall is a fall of debris, which is composed of detrital fragments prior to failure. Rapp (*2.131*, p. 104) suggested that falls of newly detached material be called primary and those involving earlier transported loose debris, such as that from shelves, be called secondary. Among those termed debris falls here, Rapp (*2.131*, p. 97) also distinguished pebble falls (size less than 20 mm), cobble falls (more than 20 mm, but less than 200 mm), and boulder falls (more than 200 mm). Included within falls would be the raveling of a thin colluvial layer, as illustrated by Deere and Patton (*2.36*), and of fractured, steeply dipping weathered rock, as illustrated by Sowers (*2.162*).

The falls of loess along bluffs of the lower Mississippi River valley, described in a section on debris falls by Sharpe (*2.146*, p. 75), would be called earth falls (or loess falls) in the present classification.

Topples

Topples have been recognized relatively recently as a distinct type of movement. This kind of movement consists of the forward rotation of a unit or units about some pivot point, below or low in the unit, under the action of gravity and forces exerted by adjacent units or by fluids in cracks. It is tilting without collapse. The most detailed descriptions have been given by de Freitas and Watters (*2.37*), and some of their drawings are reproduced in Figure 2.1d1 and d2. From their studies in the British Isles, they concluded that toppling failures are not unusual, can develop in a variety of rock types, and can range in volume from 100 m^3 to more than 1 Gm^3 (130 to 1.3 billion yd^3). Toppling may or may not culminate in either falling or sliding, depending on the geometry of the failing mass and the orientation and extent of the discontinuities. Toppling failure has been pictured by Hoek (*2.61*), Aisenstein (*2.1*, p. 375), and Bukovansky, Rodríquez, and Cedrún (*2.16*) and studied in detail in laboratory experiments with blocks by Hofmann (*2.63*). Forward rotation was noted in the Kimbley copper pit by Hamel (*2.56*), analyzed in a high rock cut by Piteau and others (*2.125*), and described among the prefailure movements at Vaiont by Hofmann (*2.62*). 9

Slides

In true slides, the movement consists of shear strain and displacement along one or several surfaces that are visible or may reasonably be inferred, or within a relatively narrow zone. The movement may be progressive; that is, shear failure may not initially occur simultaneously over what eventually becomes a defined surface of rupture, but rather it may propagate from an area of local failure. The displaced mass may slide beyond the original surface of rupture onto what had been the original ground surface, which then becomes a surface of separation.

Slides were subdivided in the classification published in 1958 (*2.182*) into (a) those in which the material in motion is not greatly deformed and consists of one or a few units and (b) those in which the material is greatly deformed or consists of many semi-independent units. These subtypes were further classed into rotational slides and planar slides. In the present classification, emphasis is put on the distinction between rotational and translational slides, for that

Figure 2.3. Rock fall due to undercutting along shore of Las Vegas Bay, Lake Mead, Nevada (photograph taken February 24, 1949) (*2.182*). Rock is Muddy Creek formation (Pliocene) consisting here of siltstone overlain by indurated breccia.

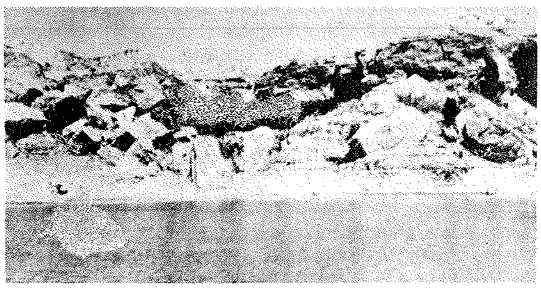
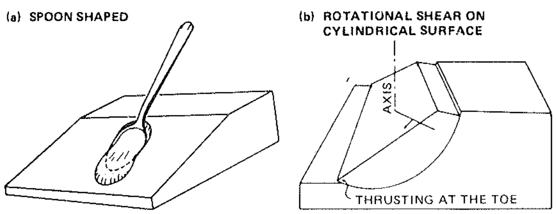


Figure 2.4. Slope failure in uniform material (*2.182*).



10 difference is of at least equal significance in the analysis of stability and the design of control methods. An indication of degree of disruption is still available by use of the terms block or intact for slides consisting of one or a few moving units and the terms broken or disrupted for those consisting of many units; these terms avoid a possible source of confusion, pointed out by D. H. Radbruch-Hall, in the use of the term debris slide, which is now meant to indicate only a slide originating in debris material, which may either proceed as a relatively unbroken block or lead to disruption into many units, each consisting of debris.

Rotational Slides

The commonest examples of rotational slides are little-deformed slumps, which are slides along a surface of rupture that is curved concavely upward. Slumps, and slumps combined with other types of movement, make up a high proportion of landslide problems facing the engineer. The movement in slumps takes place only along internal slip surfaces. The exposed cracks are concentric in plan and concave toward the direction of movement. In many slumps the underlying surface of rupture, together with the exposed scarps, is spoon-shaped (Figure 2.4). If the slide extends for a considerable distance along the slope perpendicular to the direction of movement, much of the rupture surface may approach the shape of a sector of a cylinder whose axis is parallel to the slope (Figure 2.4). In slumps, the movement is more or less rotational about an axis that is parallel to the slope. In the head area, the movement may be almost wholly downward and have little apparent rotation; however, the top surface of each unit commonly tilts backward toward the slope (Figures 2.1g, 2.1i, 2.4, 2.5, 2.6, and 2.7), but some blocks may tilt forward.

Figure 2.6 shows some of the commoner varieties of slump failure in various kinds of materials. Figure 2.7 shows the backward tilting of strata exposed in a longitudinal section through a small slump in lake beds. Although the rupture surface of slumps is generally concave upward, it is seldom a spherical segment of uniform curvature. Often the shape of the surface is greatly influenced by faults, joints, bedding, or other preexisting discontinuities of the material. The influence of such discontinuities must be considered carefully when the engineer makes a slope-stability analysis that assumes a certain configuration for the surface of rupture. Figures 2.7 and 2.8 show how the surface of rupture may follow bedding planes for a considerable part of its length. Upward thrusting and slickensides along the lateral margin of the toe of a slump are shown in Figure 2.9.

The classic purely rotational slump on a surface of smooth curvature is relatively uncommon among the many types of gravitational movement to which geologic materials are subject. Since rotational slides occur most frequently in fairly homogeneous materials, their incidence among constructed embankments and fills, and hence their interest to engineers, has perhaps been high relative to other types of failure, and their methods of analysis have in the past been more actively studied. Geologic materials are seldom uniform, however, and natural slides tend to be complex or at least significantly controlled in their mode of movement by internal inhomogeneities and discontinuities. Moreover, deeper and deeper artificial cuts for damsites, highways, and other engineering works have increasingly produced failures not amenable to analysis by the methods appropriate to circular arc slides and have made necessary the development of new methods of analytical design for prevention or cure of failures in both bedrock and engineering soils.

The scarp at the head of a slump may be almost vertical. If the main mass of the slide moves down very far, the steep scarp is left unsupported and the stage is set for a new failure (similar to the original slump) at the crown of the slide. Occasionally, the scarps along the lateral margins of the upper part of the slide may also be so high and steep that slump blocks break off along the sides and move downward and inward toward the middle of the main slide. Figure 2.10 (*2.183*) shows a plan view of slump units along the upper margins of a slide; the longest dimensions of these units are parallel with, rather than perpendicular to, the direction of movement of the main slide. Any water that finds its way into the head of a slump may be ponded by the backward tilt of the unit blocks or by other irregularities in topography so that the slide is kept wet constantly. By the successive creation of steep scarps and trapping of water, slumps often become self-perpetuating areas of instability and may continue to move and enlarge intermittently until a stable slope of very low gradient is attained.

Translational Slides

In translational sliding the mass progresses out or down and out along a more or less planar or gently undulatory surface and has little of the rotary movement or backward tilting characteristic of slump. The moving mass commonly slides out on the original ground surface. The distinction between rotational and translational slides is useful in planning control measures. The rotary movement of a slump, if the surface of rupture dips into the hill at the foot of the slide, tends to restore equilibrium in the unstable mass; the driving moment during movement decreases and the slide may stop moving. A translational slide, however, may progress indefinitely if the surface on which it rests is sufficiently inclined and as long as the shear resistance along this surface remains lower than the more or less constant driving force. A translational slide in which the moving mass consists of a single unit that is not greatly deformed or a few closely related units may be called a block slide. If the moving mass consists of many semi-independent units, it is termed a broken or disrupted slide.

The movement of translational slides is commonly controlled structurally by surfaces of weakness, such as faults, joints, bedding planes, and variations in shear strength between layers of bedded deposits, or by the contact between firm bedrock and overlying detritus (Figure 2.11). Several examples of block slides are shown in Figures 2.1j2, 2.11, 2.12, 2.13 (*2.136*), 2.14 (*2.107*), and 2.15. In many translational slides, the slide mass is greatly deformed or breaks up into many more or less independent units. As deformation and disintegration continue, and especially as water content or velocity or both increase, the broken or disrupted slide mass may change into a flow; however, all gradations exist. Broken translational slides of rock are shown in Figure 2.1j3 and of debris in Figures 2.1k and 2.16 (*2.83*).

Lateral Spreads

In spreads, the dominant mode of movement is lateral extension accommodated by shear or tensile fractures. Two types may be distinguished.

1. Distributed movements result in overall extension

Figure 2.5. Varieties of slump (*2.182*).

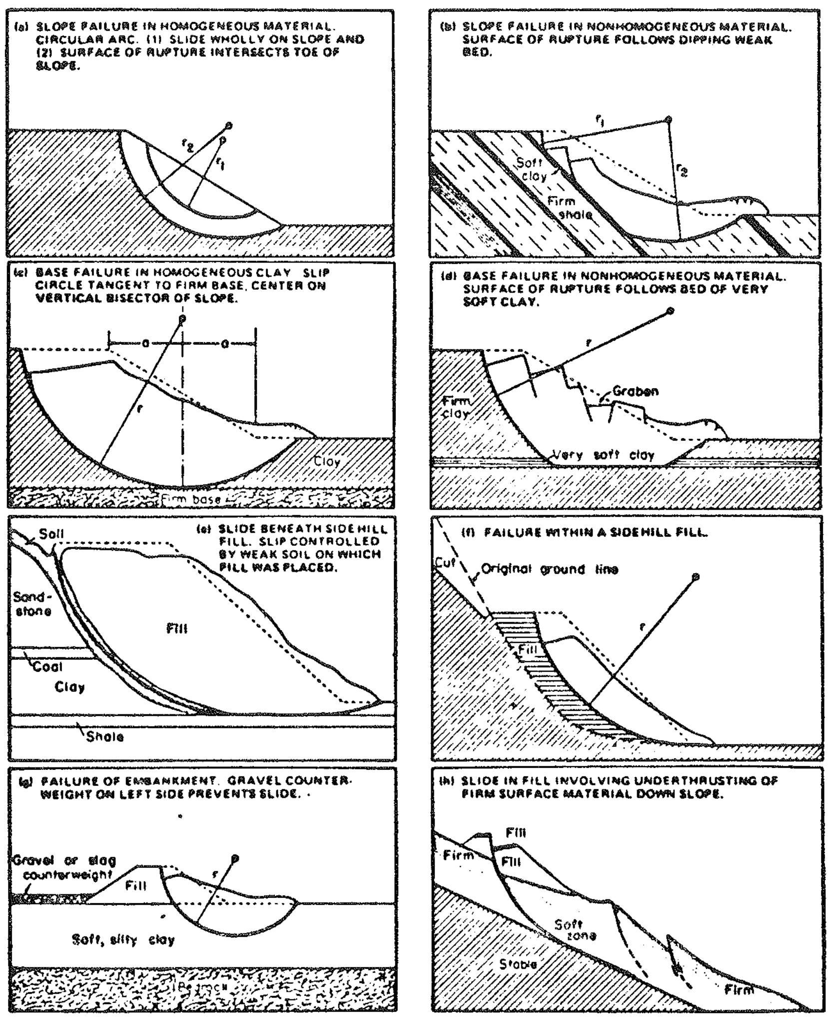


Figure 2.6. Slump of fill, controlled in this instance by failure in underlying soil (2.182).

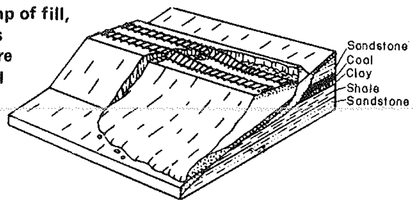


Figure 2.7. Slump in thinly bedded lake deposits of silt and clay in Columbia River valley (note backward tilting of beds above surface of rupture) (2.182).

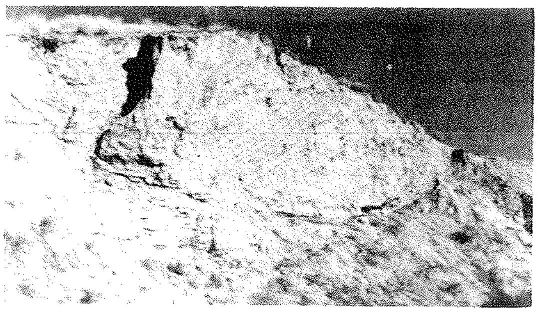


Figure 2.8. Slump in bedded deposits similar to those shown in Figure 2.7 (note that surface of rupture follows horizontal bedding plane for part of its length) (2.182).

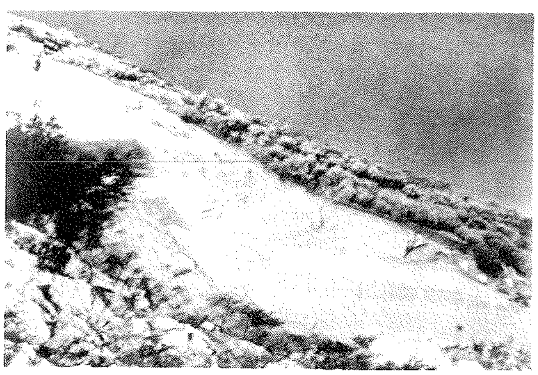
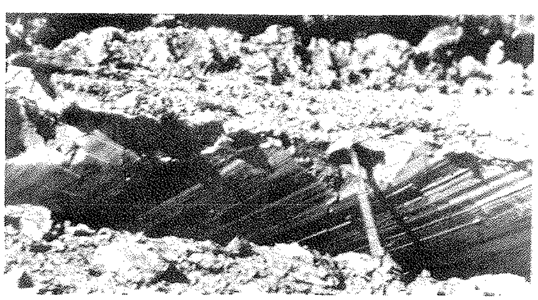


Figure 2.9. Slickensides in foot area of shallow slide in Pennington shale residuum (highly weathered clay shale) along I-40 in Roane County, Tennessee.

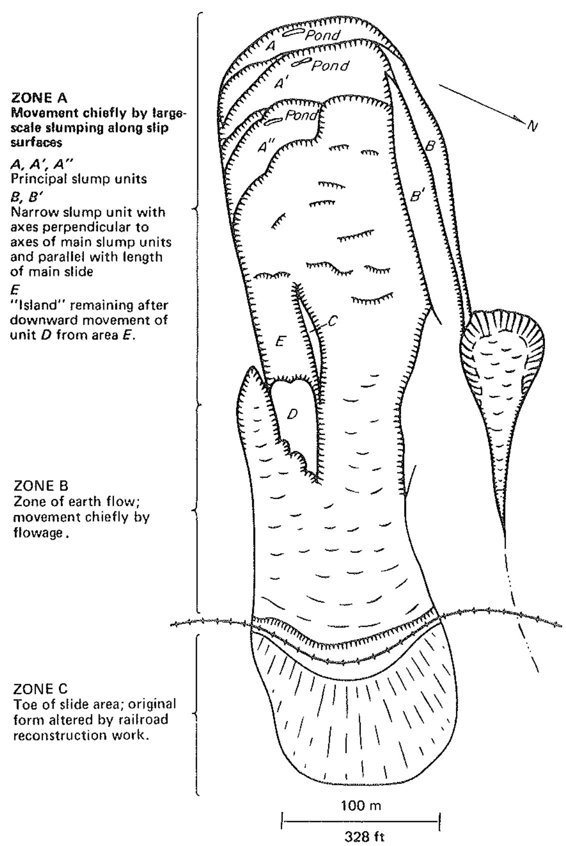


but without a recognized or well-defined controlling basal shear surface or zone of plastic flow. These appear to occur predominantly in bedrock, especially on the crests of ridges (Figure 2.1m1). The mechanics of movement are not well known.

2. Movements may involve fracturing and extension of coherent material, either bedrock or soil, owing to liquefaction or plastic flow of subjacent material. The coherent upper units may subside, translate, rotate, or disintegrate, or they may liquefy and flow. The mechanism of failure can involve elements not only of rotation and translation but also of flow; hence, lateral spreading failures of this type may be properly regarded as complex. They form, however, such a distinctive and dominant species in certain geologic situations that specific recognition seems worthwhile.

Examples of the second type of spread in bedrock are shown in Figure 2.1m2 and 2.1m3. In both examples, taken from actual landslides in the USSR and Libya respectively, a thick layer of coherent rock overlies soft shale and

Figure 2.10. Ames slide near Telluride, Colorado (2.182, 2.183). This slump-earth flow landslide occurred in glacial till overlying Mancos shale. Repeated slumping took place along upper margins after main body of material had moved down. Long axes of slump blocks *B* and *B'* are parallel with rather than perpendicular to direction of movement of main part of slide. Blocks *B* and *B'* moved toward left, rather than toward observer.



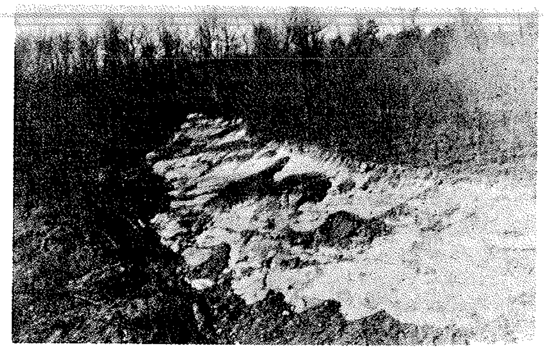


Figure 2.11. Thin layer of residual debris that slid on inclined strata of metasiltstone along I-40 in Cocke County, Tennessee.

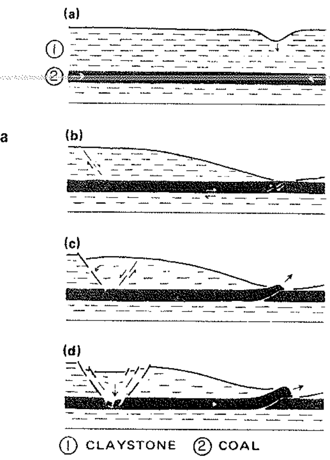


Figure 2.13. Development of landslides in horizontal sequence of claystone and coal caused by relaxation of horizontal stresses resulting from reduction in thickness of overlying strata (*2.136*).

Figure 2.12. Block slide at quarry (*2.182*).

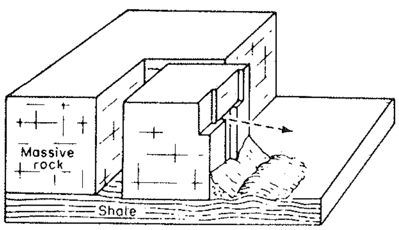
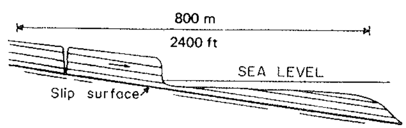


Figure 2.14. Section view of translational slide at Point Fermin, near Los Angeles, California (see also Figure 2.15) (*2.107, 2.182*). Maximum average rate of movement was 3 cm/week (1.2 in).



claystone. The underlying layer became plastic and flowed to some extent, allowing the overlying firmer rock to break into strips and blocks that then became separated. The cracks between the blocks were filled with either soft material squeezed up from below or detritus from above. The lateral extent of these slides is remarkable, involving bands several to many kilometers wide around the edges of plateaus and escarpments. The rate of movement of most lateral spreads in bedrock is apparently extremely slow.

Laterally spreading slope movements also form in fine-grained earth material on shallow slopes, particularly in sensitive silt and clay that loses most or all of its shear strength on disturbance or remolding. The failure is usually progressive; that is, it starts in a local area and spreads. Often the initial failure is a slump along a stream bank or shore, and the progressive failure extends retrogressively back from the initial failure farther and farther into the bank. The principal movement is translation rather than rotation. If the underlying mobile zone is thick, the blocks at the head may sink downward as grabens, not necessarily with backward rotation, and there may be upward and outward extrusion and flow at the toe. Movement generally begins suddenly, without appreciable warning, and proceeds with rapid to very rapid velocity.

These types appear to be members of a gradational series of landslides in surficial materials ranging from block slides at one extreme, in which the zone of flow beneath the sliding mass may be absent or very thin, to earth flows or completely liquefied mud flows at the other extreme, in which the zone of flow includes the entire mass. The form that is taken depends on local factors. Most of the larger landslides in glacial sediments of northern North America and Scandinavia lie somewhere within this series.

Lateral spreads in surficial deposits have been destructive of both life and property and have, therefore, been the subject of intensive study. Examples may be cited from Sweden (Caldenius and Lundstrom, *2.18*), Canada (Mitchell and Markell, *2.108*), Alaska (Seed and Wilson, *2.144*), and California (Youd, *2.191*). Most of the spreading failures in the western United States generally involve less than total liquefaction and seem to have been mobilized only by seismic shock. For example, there were damaging failures in San Fernando Valley, California, during the 1969 earthquake because of liquefaction of underlying sand and silt and spreading of the surficial, firmer material. The spreading failure of Bootlegger Cove clay beneath the Turnagain Heights residential district at Anchorage, Alaska, during the 1964 great earthquake resulted in some loss of life and extensive damage. In some areas within the city of San Francisco, the principal damage due to the 1906 earthquake resulted from spreading failures that not only did direct damage to structures but also severed principal water-supply lines and thereby hindered firefighting.

All investigators would agree that spreading failures in glacial and marine sediments of Pleistocene age present some common and characteristic features: Movement often occurs for no apparent external reason, failure is generally sudden, gentle slopes are often unstable, dominant movement is translatory, materials are sensitive, and pore-water pressure is important in causing instability. All de-

Figure 2.15. Translational slide at Point Fermin, California. Photograph, which was taken January 17, 1965, indicates minor slumping into gap at rear of main mass and imminent rock falls at sea cliff. Principal motion, however, was by sliding along gently seaward dipping strata.

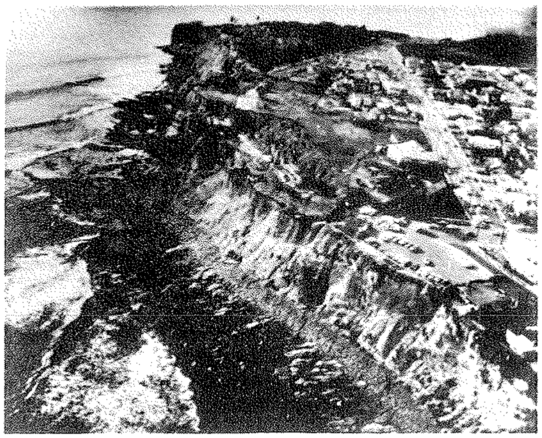
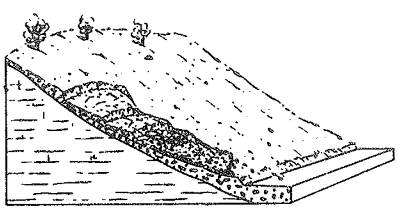


Figure 2.16. Debris slide of disintegrating soil slip variety (*2.83, 2.182*).



grees of disturbance of the masses have been observed; some failures consist almost entirely of one large slab or "flake," but others liquefy almost entirely to small chunks or mud.

Flows

Many examples of slope movement cannot be classed as falls, topples, slides, or spreads. In unconsolidated materials, these generally take the form of fairly obvious flows, either fast or slow, wet or dry. In bedrock, the movements most difficult to categorize include those that are extremely slow and distributed among many closely spaced, noninterconnected fractures or those movements within the rock mass that result in folding, bending, or bulging. In many instances, the distribution of velocities resembles that of viscous fluids; hence, the movements may be described as a form of flow of intact rock.

Much of what is here described under flowlike distributed movements has been classified as creep, both of rock and soil. But creep has come to mean different things to different persons, and it seems best to avoid the term or to use it in a well-defined manner. As used here, creep is considered to have a meaning similar to that used in mechanics of materials; that is, creep is simply deformation that continues under constant stress. Some of the creep deformation may be recoverable over a period of time upon release of the stress, but generally most of it is not. The movement commonly is imperceptible (which is usually one of the essential attributes of creep as defined in geomorphology), but increasingly sophisticated methods of measurement make this requirement difficult to apply. Furthermore, the usual partition of creep into three stages—primary (decelerating), secondary (steady or nearly so), and tertiary (accelerating to failure)—certainly includes perceptible deformation in the final stages. Laboratory studies show that both soil and rock, as well as metals, can exhibit all three stages of creep. Observations in the field, such as those reported by Müller (*2.112*) at Vaiont, embrace within the term creep perceptible movements that immediately preceded catastrophic failure.

There is disagreement also as to whether creep in rock and soil should be restricted to those movements that are distributed through a mass rather than along a defined fracture. Authorities are about equally divided on this point but, in keeping with the use of the term in engineering mechanics, the acceptance of this restriction is not favored. Creep movements can occur in many kinds of topples, slides, spreads, and flows, and the term creep need not be restricted to slow, spatially continuous deformation. Therefore, spatially continuous deformations are classified as various types of flow in rock, debris, and earth.

Flows in Bedrock

Flow movements in bedrock include deformations that are distributed among many large or small fractures, or even microfractures, without concentration of displacement along a through-going fracture. The movements are generally extremely slow and are apparently more or less steady in time, although few data are available. Flow movements may result in folding, bending, bulging, or other manifestations of plastic behavior, as shown in Figure 2.1p1, 2.1p2, 2.1p3, and 2.1p4. The distribution of velocities may roughly simulate that of viscous fluids, as shown in Figure 2.1p5.

These kinds of movements have come under close study only within the last decade or so and are being recognized more and more frequently in areas of high relief in many parts of the world. They are quite varied in character, and several kinds have been described as creep by Nemčok, Pašek, and Rybář (*2.116*) in a general classification of landslides and other mass movements, as gravitational slope deformation by Nemčok (*2.114, 2.115*), by the term Sackung (approximate translation: sagging) by Zischinsky (*2.194, 2.195*), as depth creep of slopes by Ter-Stepanian (*2.172*), and as gravitational faulting by Beck (*2.5*). In the United States, ridge-top depressions due to large-scale creep have been described by Tabor (*2.166*). A review of gravitational creep (mass rock creep) together with descriptions of examples from the United States and other countries has been prepared by Radbruch-Hall (*2.130*). The significance of these relatively slow but pervasive movements to human works on and within rock slopes is only beginning to be appreciated.

Flows in Debris and Earth

Distributed movements within debris and earth are often more accurately recognized as flows than those in rocks because the relative displacements within the mass are commonly larger and more closely distributed and the general appearance is more obviously that of a body that has behaved like a fluid. Moreover, the fluidizing effect of water itself is, as a rule, a part of the process. Slip surfaces within the moving mass are usually not visible or are short lived, and the boundary between moving mass and material in place may be a sharp surface of differential movement or a zone of distributed shear.

There is complete gradation from debris slides to debris flows, depending on water content, mobility, and character of the movement, and from debris slide to debris avalanche as movement becomes much more rapid because of lower cohesion or higher water content and generally steeper slopes. Debris slides and, less commonly, debris avalanches may have slump blocks at their heads. In debris slides, the moving mass breaks up into smaller and smaller parts as it advances toward the foot, and the movement is usually slow. In debris avalanches, progressive failure is more rapid, and the whole mass, either because it is quite wet or because it is on a steep slope, liquefies, at least in part, flows, and tumbles downward, commonly along a stream channel, and may advance well beyond the foot of the slope. Debris avalanches are generally long and narrow and often leave a serrate or V-shaped scar tapering uphill at the head, as shown in Figures 2.1q3 and 2.17, in contrast to the horseshoe-shaped scarp of a slump.

Debris flows, called mud flows in some other classifications, are here distinguished from the latter on the basis of particle size. That is, the term debris denotes material that contains a relatively high percentage of coarse fragments, whereas the term mud flow is reserved for an earth flow consisting of material that is wet enough to flow rapidly and that contains at least 50 percent sand-, silt-, and clay-sized particles. Debris flows commonly result from unusually heavy precipitation or from thaw of snow or frozen soil. The kind of flow shown in Figure 2.1q1 often occurs during torrential runoff following cloudbursts. It is favored by the presence of soil on steep mountain slopes from which the vegetative cover has been removed by fire or other means, but the absence of vegetation is not a prerequisite. Once in motion, a small stream of water heavily laden with soil has transporting power that is disproportionate to its size, and, as more material is added to the stream by caving of its banks, its size and power increase. These flows commonly follow preexisting drainageways, and they are often of high density, perhaps 60 to 70 percent solids by weight, so that boulders as big as automobiles may be rolled along. If such a flow starts on an unbroken hillside it will quickly cut a V-shaped channel. Some of the coarser material will be heaped at the site to form a natural levee, while the more fluid part moves down the channel (Figure 2.17). Flows may extend many kilometers, until they drop their loads in a valley of lower gradient or at the base of a mountain front. Some debris flows and mud flows have been reported to proceed by a series of pulses in their lower parts; these pulses presumably are caused by periodic mobilization of material in the source area or by periodic damming and release of debris in the lower channel.

The term avalanche, if unmodified, should refer only to

Figure 2.17. Debris avalanche or very rapid debris flow at Franconia Notch, New Hampshire, June 24, 1948, after several days of heavy rainfall (*2.182*). Only soil mantle 2 to 5 m (7 to 6 ft) thick, which lay over bedrock on a slope of about 1:1, was involved. Scar is about 450 m (1500 ft) long; natural levees can be seen along sides of flow. US-3 is in foreground.

slope movements of snow or ice. Rapp (*2.132*) and Temple and Rapp (*2.169*), with considerable logic, recommend that, because the term debris avalanche is poorly defined, it should be abandoned, and that the term avalanche should be used only in connection with mass movements of snow, either pure or mixed with other debris. The term debris avalanche, however, is fairly well entrenched and in common usage (Knapp, *2.86*); hence, its appearance in the classification as a variety of very rapid to extremely rapid debris flow seems justified.

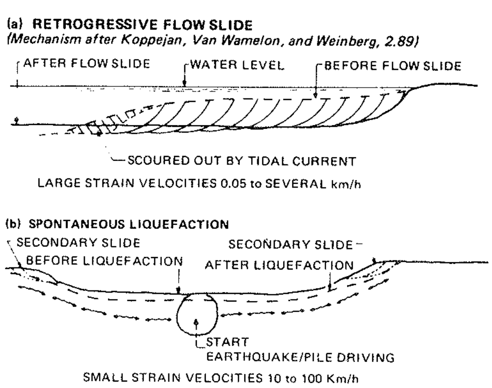
Recent studies have contributed much to a better understanding of the rates and duration of rainfall that lead to the triggering of debris flows, the physical properties of the material in place, the effect of slope angle, the effect of pore-water pressure, the mobilization of material and mechanism of movement, and the properties of the resulting deposit. The reader is referred especially to the works of Campbell (*2.20*), Daido (*2.34*), Fisher (*2.46*), Hutchinson (*2.70*), Hutchinson and Bhandari (*2.72*), Johnson and Rahn (*2.76*), Jones (*2.78*), Prior, Stephens, and Douglas (*2.129*), Rapp (*2.131*), K. M. Scott (*2.141*), R. C. Scott (*2.142*), and Williams and Guy (*2.188*). Flowing movements of surficial debris, including creep of the mantle of weathered rock and soil, are shown in Figure 2.1q2, 2.1q4, and 2.1q5. Soil flow, or solifluction, which in areas of perennially or permanently frozen ground is better termed gelifluction, takes many forms and involves a variety of mechanisms that can be treated adequately only in works devoted to this special field, which is of great significance to engineering works at high latitudes and altitudes. The reader is referred to summaries by Dylik (*2.40*), Washburn (*2.187*), and Corte (*2.27*); the proceedings of the International Conference on Permafrost (*2.111*); and recent work by McRoberts and Morgenstern (*2.104, 2.105*) and Embleton and King (*2.42*).

Subaerial flows in fine-grained materials such as sand, silt, or clay are classified here as earth flows. They take a variety of forms and range in water content from above saturation to essentially dry and in velocity from extremely rapid to extremely slow. Some examples are shown in Figure 2.1r1 through 2.1r5. At the wet end of the scale are mud flows, which are soupy end members of the family of predominantly fine-grained earth flows, and subaqueous flows or flows originating in saturated sand or silt along shores.

In a recent paper reviewing Soviet work on mud flows, Kurdin (*2.91*) recommended a classification of mud flows based on (a) the nature of the water and solid-material supply; (b) the structural-rheological model, that is, whether the transporting medium is largely water in the free state or is a single viscoplastic mass of water and fine particles; (c) the composition of the mud flow mass, that is, whether it consists of mud made up of water and particles less than 1 mm (0.04 in) in size or of mud plus gravel, rubble, boulders, and rock fragments; and (d) the force of the mud flow as defined by volume, rate of discharge, and observed erosive and destructive power. In the Soviet literature mud flows include not only what are here classified as debris flows but also heavily laden flows of water-transported sediment.

According to Andresen and Bjerrum (*2.3*), subaqueous flows are generally of two types: (a) retrogressive flow slide or (b) spontaneous liquefaction, as shown in Figure 2.18. The retrogressive flows, as shown in Figure 2.1r1, occur mostly along banks of noncohesive clean sand or silt. They are especially common along tidal estuaries in the coastal provinces of Holland, where banks of sand are subject to scour and to repeated fluctuations in pore-water pressure because of the rise and fall of the tide (Koppejan,

Figure 2.18. Retrogressive flow slide and spontaneous liquefaction (*2.3*).



Note: 1 km/h = 0.6 mph.

Figure 2.19. Earth flow near Greensboro, Florida (*2.80, 2.182*). Material is flat-lying, partly indurated clayey sand of the Hawthorn formation (Miocene). Length of slide is 275 m (900 ft) from scarp to edge of trees in foreground. Vertical distance is about 15 m (45 ft) from top to base of scarp and about 20 m (60 ft) from top of scarp to toe. Slide occurred in April 1948 after year of unusually heavy rainfall, including 40 cm (16 in) during 30 d preceding slide.

Van Wamelon, and Weinberg, *2.89*). When the structure of the loose sand breaks down along a section of the bank, the sand flows rapidly along the bottom, and, by repeated small failures, the slide eats into the bank and enlarges the cavity. Sometimes the scarp produced is an arc, concave toward the water, and sometimes it enlarges greatly, retaining a narrow neck or nozzle through which the sand flows. An extensive discussion and classification of subaqueous mass-transport processes and the resulting deposits have been presented by Carter (*2.21*).

Rapid earth flows also occur in fine-grained silt, clay, and clayey sand, as shown in Figures 2.1r2 and 2.19 (*2.80*). These flows form a complete gradation with slides involving failure by lateral spreading, but they involve not only liquefaction of the subjacent material but also retrogressive failure and liquefaction of the entire slide mass. They usually take place in sensitive materials, that is, in those materials whose shear strength on remolding at constant water content is decreased to a small fraction of its original value. Rapid earth flows have caused loss of life and immense destruction of property in Scandinavia, the St. Lawrence River valley in Canada, and Alaska during the 1964 earthquake. The properties of the material involved, which is usually a marine or estuarine clay of late Pleistocene age, have been thoroughly studied by many investigators during the last 15 years. Summary papers have been written by Bjerrum and others (*2.12*) on flows in Norway and by Mitchell and Markell (*2.108*), and Eden and Mitchell (*2.41*) on flows in Canada. Shoreline flows produced by the Alaskan earthquake at Valdez and Seward have been described by Coulter and Migliaccio (*2.28*) and Lemke (*2.98*). The large failure on the Reed Terrace near Kettle Falls, Washington, shown in Figures 2.20 and 2.21 (*2.79*), resembles in some respects the earth flow at Riviere Blanche, Quebec, shown in Figure 2.1r2 (*2.146*).

The somewhat drier and slower earth flows in plastic earth are common in many parts of the world wherever there is a combination of clay or weathered clay-bearing rocks, moderate slopes, and adequate moisture; Figure 2.22 shows a typical example. A common elongation of the flow, channelization in depression in the slope, and spreading of the toe are illustrated in Figure 2.1r3 and also shown in an actual debris flow in Figure 2.23.

The word flow naturally brings water to mind, and some content of water is necessary for most types of flow movement. But small dry flows of granular material are common, and a surprising number of large and catastrophic flow movements have occurred in quite dry materials. Therefore, the classification of flows indicates the complete range of water content—from liquid at the top to dry at the bottom. Tongues of rocky debris on steep slopes moving extremely slowly and often fed by talus cones at the head are called block streams (Figure 2.1q5). Because of rainwash, a higher proportion of coarse rocks may be in the surface layers than in the interior. Dry flows of sand are common along shores or embankments underlain by dry granular material. In form, they may be channelized, as shown in Figures 2.1r4 and 2.24, or sheetlike, as shown in Figure 2.25 (*2.79*). Small flows of dry silt, powered by impact on falling from a cliff, have been recognized, but so far as is known none has been studied in detail (Figure 2.26).

Flows of loess mobilized by earthquake shock have been more destructive of life than any other type of slope failure. Those that followed the 1920 earthquake in Kansu Province, China (Close and McCormick, *2.23*), shown in Figure 2.1r5, took about 100 000 lives. Apparently the normal, fairly coherent internal structure of the porous silt was destroyed by earthquake shock, so that, for all practical purposes, the loess became a fluid suspension of silt in air and flowed down into the valleys, filling them and overwhelming villages. The flows were essentially dry, according to the report. Extensive flows of loess accompanied the Chait earthquake of July 10, 1949, in Tadzhikistan, south-central Asia, and buried or destroyed 33 villages as the flows covered the bottoms of valleys to depths of several tens of meters for many kilometers (Gubin, *2.54*).

Complex

More often than not, slope movements involve a combina-

Figure 2.20. Reed Terrace area, right bank of Lake Roosevelt reservoir on Columbia River, near Kettle Falls, Washington, May 15, 1951 (*2.182*). Landslide of April 10, 1952, involving about 11 Mm^3 (15 million yd^3) took place by progressive slumping, liquefaction, and flowing out of glaciofluvial sediments through narrow orifice into bottom of reservoir.

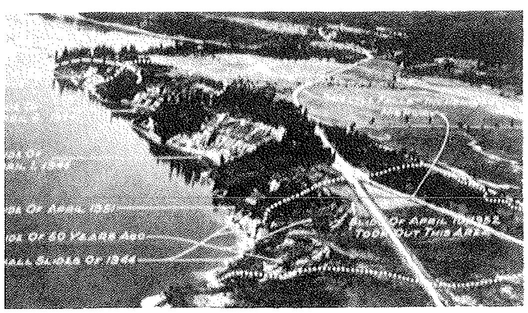


Figure 2.21. Reed Terrace area, Lake Roosevelt, Washington, August 1, 1952, after landslide of April 10, 1952 (*2.79, 2.182*).

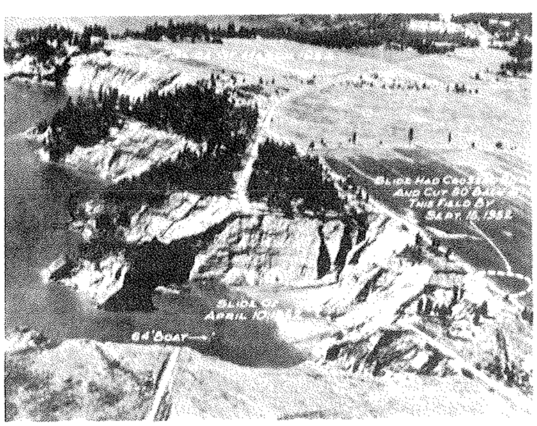


Figure 2.22. Earth flow developing from slump near Berkeley, California (*2.182*).

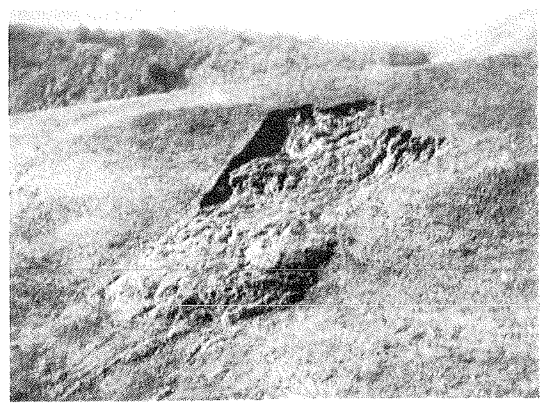


Figure 2.23. Old debris flow in altered volcanic rocks west of Pahsimeroi River in south central Idaho.

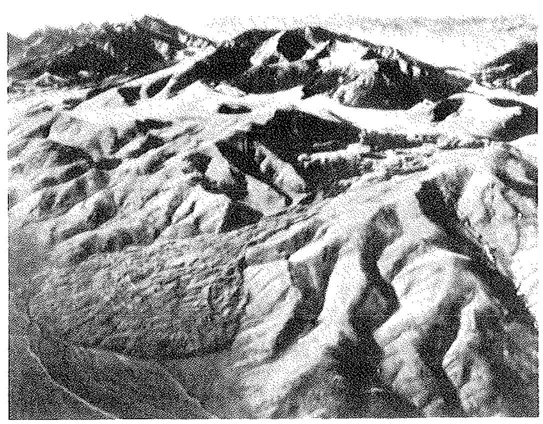


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Figure 2.24. Dry sand flow in Columbia River valley (*2.182*). Material is sand over lake-bed silt; dry sand from upper terrace flowed like liquid through notch in more compact sand and silt below.

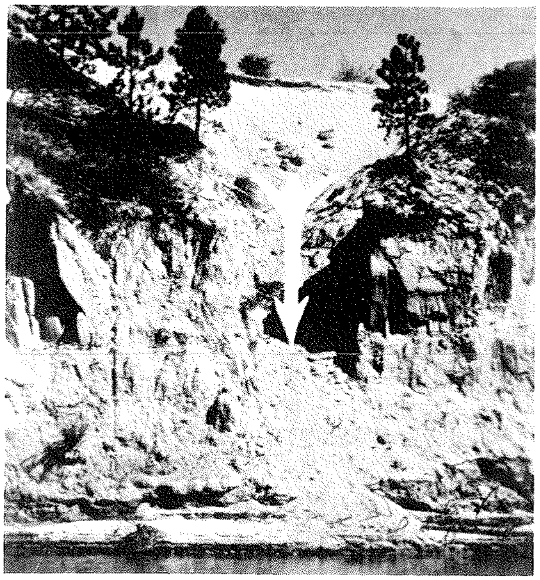
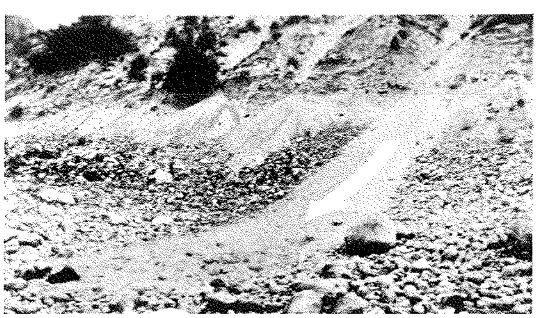


Figure 2.25. Shallow, dry, sand flow along shore of Lake Roosevelt, Washington (*2.79*). Wave erosion or saturation of sediment by lake water caused thin skin of material to lose support and ravel off terrace scarp.

Figure 2.26. Dry flow of silt (*2.182*). Material is lake-bed silt of Pleistocene age from high bluff on right bank of Columbia River, 4 km (2.5 miles) downstream from Belvedere, Washington. Flow was not observed while in motion, but is believed to result from blocks of silt falling down slope, disintegrating, forming a single high-density solid-in-air suspension, and flowing out from base of cliff.



tion of one or more of the principal types of movement described above, either within various parts of the moving mass or at different stages in development of the movements. These are termed complex slope movements, and a few examples of the many possible types are illustrated in Figure 2.1s1 through 2.1s5.

Of particular interest regarding hazards of landslides to life and property are large, extremely rapid rock fall-debris flows, referred to as rock-fragment flow (variety rock-fall avalanche) in the 1958 classification (*2.182*). Rock slide- and rock fall-debris flows are most common in rugged mountainous regions. The disaster at Elm, Switzerland (Heim, *2.58*, pp. 84, 109-112), which took 115 lives, started with small rock slides at each side of a quarry on the mountainside. A few minutes later the entire mass of rock above the quarry crashed down and shot across the valley. The movement of the rock fragments, which had to that moment been that of a rock slide and rock fall, appears to have taken on the character of a flow. The mass rushed up the other side of the small valley, turned and streamed into the main valley, and flowed for nearly 1.5 km (1 mile) at high velocity before stopping (Figure 2.1s1). About 10 Mm^3 (13 million yd^3) of rock descended an average of 470 m (1540 ft) vertically in a total elapsed time of about 55 s. The kinetic energy involved was enormous. A similar and even larger rock-fall avalanche occurred at Frank, Alberta, in 1903 and also caused great loss of life and property (McConnell and Brock, *2.103*; Cruden and Krahn, *2.33*).

These rock fall-debris flows are minor, however, compared with the cataclysmic flow that occurred at the time of the May 31, 1970, earthquake in Peru, which buried the city of Yungay and part of Ranrahirca, causing a loss of more than 18 000 lives. According to Plafker, Ericksen, and Fernandez Concha (*2.126*), the movement started high on Huascarán Mountain at an altitude of 5500 to 6400 m and involved 50 Mm^3 to 100 Mm^3 (65 million to 130 million yd^3) of rock, ice, snow, and soil that traveled 14.5 km (9 miles) from the source to Yungay at a velocity between 280 and 335 km/h (175 to 210 mph). They reported strong evidence that the extremely high velocity and low friction of the flow were due, at least in part, to lubrication by a cushion of air entrapped beneath the debris. Pautre, Sabarly, and Schneider (*2.122*) suggested that the mass may have ridden on a cushion of steam. A sketch of the area affected is shown in Figure 2.27, taken from a paper by Cluff (*2.24*) on engineering geology observations. Crandell and Fahnestock (*2.29*) cited evidence for an air cushion beneath one or more rock fall-debris flows that occurred in December 1963 at Little Tahoma Peak and Emmons Glacier on the east flank of Mt. Rainier volcano, Washington.

Such flows probably cannot be produced by a few thousand or a few hundred thousand cubic meters of material. Many millions of megagrams are required; and, when that much material is set in motion, perhaps even slowly, predictions of behavior based on past experience with small failures become questionable. The mechanics of large, extremely rapid debris flows, many of which appear to have been nearly dry when formed, have come under much recent study. The large prehistoric Blackhawk landslide (Figure 2.28) shows so little gross rearrangement within the sheet of debris of which it is composed that Shreve (*2.148*) believed the broken material was not fluidized but slid on an ephemeral layer of compressed air. He reported, similarly, that the large landslide that was triggered by the Alaska earthquake of 1964 and fell onto the Sherman Glacier showed little large-scale mixing and did not flow like a viscous fluid but instead slid like a flexible sheet (Shreve, *2.149*). On the other hand, Johnson and Ragle (*2.77*) reported,

Many rock-snow slides that followed from the Alaska earthquake of March 27, 1964, illustrated a variety of flow mechanics. The form of some slides suggests a complete turbulence during flow, while the form of others gives evidence for steady-state flow or for controlled shearing.

From a detailed analysis of the kinematics of natural rock fall-debris flows and from model studies, Hsü (*2.66*) disputed Shreve's hypothesis that some slid as relatively undeformed sheets on compressed air and concluded, rather, that they flowed.

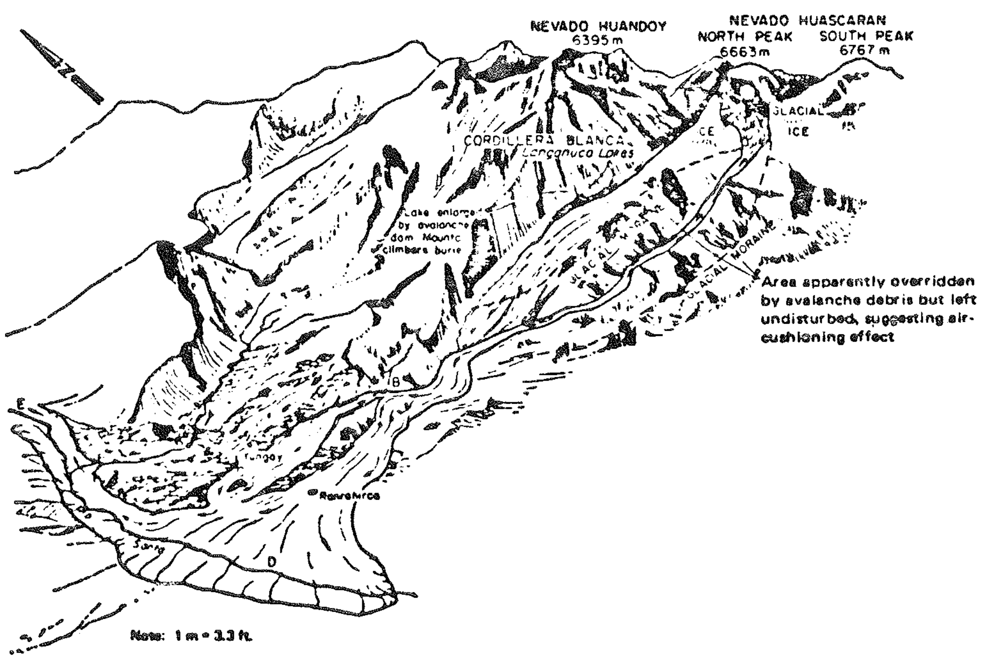
Obviously, there is much yet to be learned about these processes, particularly as similar features indicating mass movements of huge size have been recognized in Mariner 9 photographs of the surface of Mars (Sharp, *2.145*), where it is yet uncertain that significant amounts of either liquid or gas were available for fluidization.

Getting back to Earth, we note self-explanatory examples of complex movements in Figure 2-1: slump-topple in Figure 2.1s2, rock slide-rock fall in Figure 2.1s3, and the common combination of a slump that breaks down into an earth flow in its lower part in Figure 2.1s5.

The illustration of cambering and valley bulging in Figure 2.1s4 is adapted from the classical paper by Hollingworth and Taylor (*2.65*) on the Northampton Sand Ironstone in England, their earlier paper on the Kettering district (*2.64*), and a sketch supplied by J. N. Hutchinson. The complex movements were described by Hutchinson (*2.68*) as follows:

Cambering and Valley Bulging. These related features were first clearly recognized in 1944 by Hollingworth, Taylor, and Kellaway (see reference in Terzaghi, 1950) in the Northampton Ironstone field of central England, where they are believed to have a Late Pleistocene origin. The ironstone occurs in the near-horizontal and relatively thin Northampton Sands, which are the uppermost solid rocks in the neighborhood. These are underlain, conformably, by a great thickness of the Lias, into which shallow valleys, typically 1200 to 1500 meters wide and 45 meters deep, have been eroded. Excavations for dam trenches in the valley bottoms have shown the Lias there to be thrust strongly upward and contorted, while opencast workings in the Northampton Sands

Figure 2.27. Area affected by May 31, 1970, Huascarán debris avalanche, which originated at point A (*2.24*). Yungay was protected from January 10, 1962, debris avalanche by 180 to 240-m (600 to 800-ft) high ridge (point B), but a portion of May 31, 1970, debris avalanche diverted from south side of canyon wall, topped "protective" ridge, and descended on Yungay below. Only safe place in Yungay was Cemetery Hill (point C), where some 93 people managed to run to before debris avalanche devastated surrounding area. Moving at average speed of 320 km (200 mph), debris arrived at point D, 14.5 km (9 miles) distant and 3660 m (12 000 ft) lower, within 3 to 4 min after starting from north peak of Huascarán. Debris flowed upstream along course of Rio Santa (point E) approximately 2.5 km (1.5 miles). Debris continued to follow course of Rio Santa downstream to Pacific Ocean, approximately 160 km (100 miles), devastating villages and crops occupying floodplain.



occupying the interfluves reveal a general valleyward increase of dip of "camber" of this stratum, often passing into dip and fault structure, suggesting corresponding downward movements along the valley margins. In adjusting to these movements, the rigid cap-rock has been dislocated by successive, regularly spaced fissures which parallel the valley and are known as "gulls." Similar features have been recognized in other parts of England and in Bohemia. The mechanisms by which cambering and valley bulging have been formed remain to be established.

Hutchinson (*2.70*) also pointed out that Sharpe's definition of flow (*2.146*), which requires zero relative displacement at the boundary of the flow (flow adheres to the stable material), does not fit the observed distribution of velocities at Beltinge, where a mud flow developed in a temperate climate on a 30-m-high (98-ft) coastal cliff of stiff, fissured London clay subject to moderate marine erosion. Here the mud flow was bounded both on the sides and on the bottom by discrete surfaces along which shear displacements occurred. For these kinds of movements Hutchinson and Bhandari (*2.72*) proposed the term mudslides. These can be regarded as complex movements in which the internal distribution of velocities within the moving mass may or may not resemble that of viscous fluids, but the movement relative to stable ground is finite discontinuous shear. It would seem that the material of the sliding earth flow is behaving as a plastic body in plug flow, as suggested by Hutchinson (*2.69*, pp. 231-232) and as analyzed in detail by Johnson (*2.75*).

Sequence or Repetition of Movement

The term retrogressive has been used almost consistently for slides or flow failures that begin at a local area, usually along a slope, and enlarge or retreat opposite to the direction of movement of the material by spreading of the failure surface, successive rotational slumps, falls, or liquefaction of the material. Kojan, Foggin, and Rice (*2.87*, pp. 127-128) used the term for failure spreading downslope.

On the other hand, the term progressive has been used to indicate extension of the failure (a) downslope (Blong, *2.13*; Kjellman, *2.84*; Ter-Stepanian, *2.170, 2.171*; Thomson and Hayley, *2.177*), (b) upslope (but not specifically upslope only) (Seed, *2.143*; Tavenas, Chagnon, and La Rochelle, *2.168*), and (c) either upslope or downslope, or unspecified (Terzaghi and Peck, *2.176*; Bishop, *2.7*; Romani, Lovell, and Harr, *2.135*; Lo, *2.100*; Frölich, *2.51*; Ter-Stepanian and Goldstein, *2.173*, and many others).

I suggest that the term progressive be used for failure that is either advancing or retreating or both simultaneously, that the term retrogressive be used only for retreating failures, and that failures that enlarge in the direction of movement be referred to simply as advancing failures.

The terms complex, composite, compound, multiple, and successive have been used in different ways by various authors. I suggest the following definitions.

1. Complex refers to slope movements that exhibit more than one of the major modes of movement. This is the sense of the meaning suggested by Blong (*2.14*). The term is synonymous with composite, as used by Prior, Stephens, and Archer (*2.128*) to describe sliding mud flows.

Figure 2.28. Blackhawk landslide (*2.147*). Upslope view, southward over lobe of dark marble breccia spread beyond mouth of Blackhawk Canyon on north flank of San Bernardino Mountains in southern California. Maximum width of lobe is 3.2 km (2 miles); height of scarp at near edge is about 15 m (50 ft).

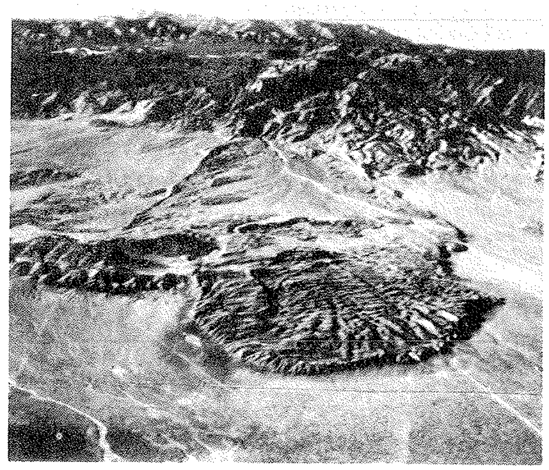
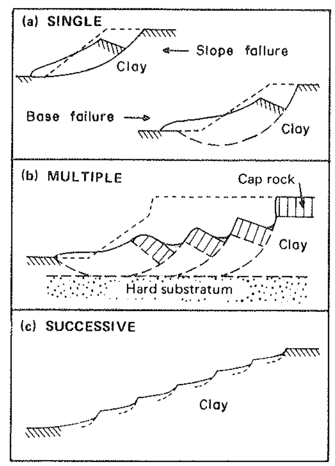


Figure 2.29. Main types of rotational slide (*2.68*).



2. Compound refers to movements in which "the failure surface is formed of a combination of curved and planar elements and the slide movements have a part-rotational, part-translational character" (Skempton and Hutchinson, *2.154*).

3. Multiple refers to manifold development of the same mode of movement. As applied to retrogressive rotational sliding, the term refers to the production of "two or more slipped blocks, each with a curved slip surface tangential to a common, generally deep-seated slip sole [Figure 2.29]. Clearly, as the number of units increases, the overall character of the slip becomes more translational, though in failing, each block itself rotates backwards" (Hutchinson, *2.68*). Leighton (*2.97*) distinguished two types of multiple slide blocks: superposed, in which each slide block rides out on the one below, and juxtaposed, in which adjacent

moving units have a common basal surface of rupture, as shown in Figure 2.30.

4. Successive refers to any type of multiple movements that develop successively in time. According to Skempton and Hutchinson (*2.154*), "Successive rotational slips consist of an assembly of individual shallow rotational slips. The rather sparse data available suggest that successive slips generally spread up a slope from its foot." Hutchinson (*2.67*) states,

> *Below a slope inclination of about 13°* [in London Clay], *rotational slips of type R are replaced by successive rotational slips (type S). These probably develop by retrogression from a type R slip in the lower slope. Each component slip is usually of considerable lateral extent, forming a step across the slope. Irregular successive slips, which form a mosaic rather than a stepped pattern in plan are also found.*

Figure 2.31 (*2.67*) shows the main types of landslides in London clay.

Landslides that develop one on top of another are called multistoried by Ter-Stepanian and Goldstein (*2.173*). Figure 2.32 shows their illustration of a three-storied landslide in Sochi on the coast of the Black Sea.

Rate of Movement

The rate-of-movement scale used in this chapter is shown at the bottom of the classification chart in Figure 2.1u. Metric equivalents to the rate scale shown in the 1958 classification have been derived by Yemel'ianova (*2.190*), and these should now be regarded as the primary definitions.

TERMS RELATING TO MATERIAL

Principal Divisions

The following four terms have been adopted as descriptions of material involved in slope movements.

1. Bedrock designates hard or firm rock that was intact and in its natural place before the initiation of movement.
2. Engineering soil includes any loose, unconsolidated, or poorly cemented aggregate of solid particles, generally of natural mineral, rock, or inorganic composition and either transported or residual, together with any interstitial gas or liquid. Engineering soil is divided into debris and earth.

a. Debris refers to an engineering soil, generally surficial, that contains a significant proportion of coarse material. According to Shroder (*2.150*), debris is used to specify material in which 20 to 80 percent of the fragments are greater than 2 mm (0.08 in) in size and the remainder of the fragments less than 2 mm.

b. Earth (again according to Shroder) connotes material in which about 80 percent or more of the fragments are smaller than 2 mm; it includes a range of materials from nonplastic sand to highly plastic clay.

This division of material that is completely gradational is admittedly crude; however, it is intended mainly to enable a name to be applied to material involved in a slope

Figure 2.30. Two types of multiple slide blocks (*2.97*).

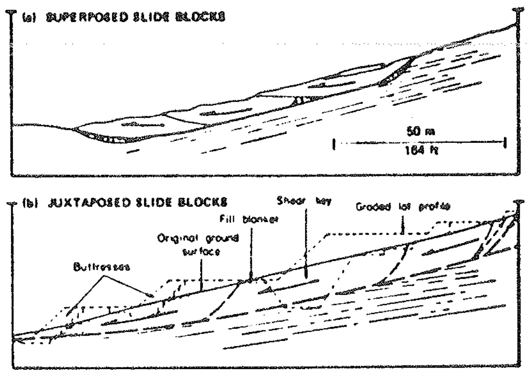
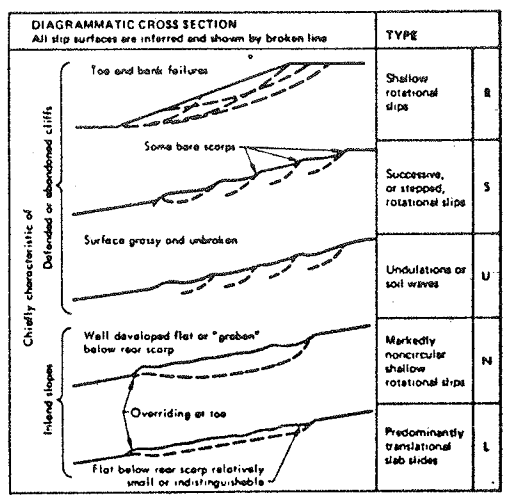


Figure 2.31. Main types of landslides in London clay (*2.67*).



movement on the basis of a limited amount of information.

Water Content

By modifying the suggestions of Radbruch-Hall (*2.130*), we may define terms relating to water content simply as (a) dry, contains no visible moisture; (b) moist, contains some water but no free water and may behave as a plastic solid but not as a liquid; (c) wet, contains enough water to behave in part as a liquid, has water flowing from it, or supports significant bodies of standing water; and (d) very wet, contains enough water to flow as a liquid under low gradients.

Texture, Structure, and Special Properties

As amounts of information increase, more definite designation can be made about slope movements. For example, a bedrock slump may be redesignated as a slump in sandstone

Figure 2.32. Three-storied landslide in Sochi on coast of Black Sea, USSR (*2.173*). Boundaries of three stories of sliding are shown in section and plan by three types of lines.

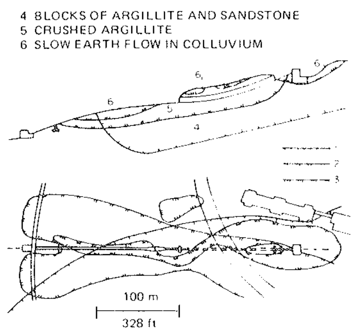
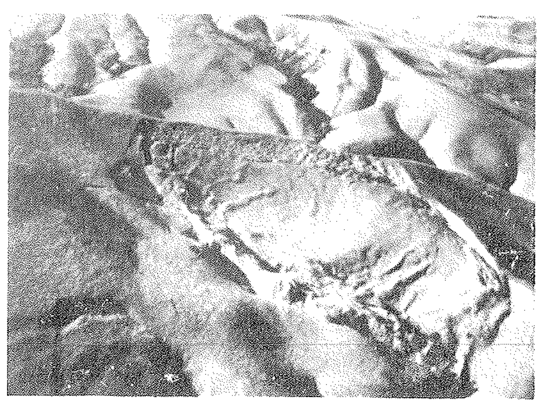


Figure 2.33. Shallow translational slide that developed on shaly slope in Puente Hills of southern California (*2.147*). Slide has low D/L ratio (note wrinkles in surface).



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over stiff-fissured clay shale, or an earth slide may be given more precise definition as a block slide in moist sensitive clay.

TERMS RELATING TO SIZE OR GEOMETRY

A rather large body of descriptive terms has been built up relating to the size, shape, and morphology of slope movements and their deposits. Some of these have already been mentioned, such as the relation of rotational slides to curved surfaces of rupture and translational slides to planar surfaces of rupture. The close association between the morphology of a slope movement and its dominant genetic process, which is evident in a qualitative way from the foregoing text and illustrations, has been tested quantitatively through the use of refined measures of the parts and geometric attributes of landslides by Crozier (*2.32*) and by Blong (*2.14, 2.15*). These authors, together with Snopko (*2.157*), Klengel and Pašek (*2.85*), Shroder (*2.150*), and Laverdière (*2.94*), have made available a terminology that is adequate to describe almost any feature of a slump earth flow. In addition, Skempton and Hutchinson (*2.154*) used the ratio of D/L, where D is the maximum thickness of the slide and L is the maximum length of the slide upslope. From Skempton's figures showing original use of this ratio (*2.152*), it seems probable that the intended length is that of a chord of the rupture surface (L_c), rather than the total length (L), as shown in Figure 2.1t. Skempton and Hutchinson gave a range of D/L_c values of 0.15 to 0.33 for rotational slides in clay and shale, and they stated that slab slides, which commonly occur in a mantle of weathered or colluvial material on clayey slopes, rarely if ever have D/L_c ratios greater than 0.1. Figure 2.33 illustrates such a shallow slab slide. In a statistical study of the forms of landslides along the Columbia River valley, Jones, Embody, and Peterson (*2.79*) made extensive use of the horizontal component (HC) or distance from the foot of the landslide to the crown, measured in a longitudinal section of the landslide, and the vertical component (VC) or difference in altitude between the foot and crown, measured in the same section.

TERMS RELATING TO GEOLOGIC, GEOMORPHIC, GEOGRAPHIC, OR CLIMATIC SETTING

The classification of landslides proposed by Savarensky (*2.140*) and followed to some degree in eastern Europe makes the primary division of types on the basis of the relation of slope movements to the geologic structure of the materials involved. Accordingly, asequent slides are those in which the surface of rupture forms in homogeneous material; consequent slides are those in which the position and geometry of the surface of rupture are controlled by preexisting discontinuities such as bedding, jointing, or contact between weathered and fresh rock; and insequent slides are those in which the surface of rupture cuts across bedding or other surfaces of inhomogeneity. The Japanese have used a classification of landslides separated into (a) tertiary type, involving incompetent tertiary sedimentary strata (Takada, *2.167*); (b) hot-spring-volcanic type, which is in highly altered rocks; and (c) fracture-zone type, which occurs in fault zones and highly broken metamorphic rocks. Sharpe (*2.146*, pp. 57-61) distinguished three types of mud flows: semiarid, alpine, and volcanic, to which Hutchinson (*2.68*) has added a fourth variety, temperate.

Types of landslides are sometimes identified by the geographic location at which the type is particularly well developed. For example, Sokolov (*2.159*) refers to block slides of the Angara type (similar to that shown in Figure 2.1m2), of the Tyub-Karagan type (similar to that shown in Figure 2.1m3), and of the Ilim and Crimean types, all named after localities. Reiche (*2.133*) applied the term Toreva-block (from the village of Toreva on the Hopi Indian Reservation in Arizona) to "a landslide consisting essentially of a single large mass of unjostled material which, during descent, has undergone backward rotation toward the parent cliff about a horizontal axis which roughly parallels it" (Figure 2.1g). Shreve (*2.149*), in summarizing data on landslides that slid on a cushion of compressed air, referred to these landslides as being of the Blackhawk type, from the rock fall-debris slide-debris flow at Blackhawk Mountain in southern California (*2.148*). Although the use of locality terms may occasionally be a convenience, it is not recommended as a general practice, for the terms themselves are not informative to a reader who lacks knowledge of the locality.

TERMS RELATING TO AGE OR STATE OF ACTIVITY

Active slopes are those that are either currently moving or that are suspended, the latter term implying that they are not moving at the present time but have moved within the last cycle of seasons. Active slides are commonly fresh; that is, their morphological features, such as scarps and ridges, are easily recognizable as being due to gravitational movement, and they have not been significantly modified by surficial processes of weathering and erosion. However, in arid regions, slides may retain a fresh appearance for many years.

Inactive slopes are those for which there is no evidence that movement has taken place within the last cycle of seasons. They may be dormant, in which the causes of failure remain and movement may be renewed, or they may be stabilized, in which factors essential to movement have been removed naturally or by human activity. Slopes that have long-inactive movement are generally modified by erosion and weathering or may be covered with vegetation so that the evidence of the last movement is obscure. They are often referred to as fossil (Záruba and Mencl, *2.193*; Klengel and Pašek, *2.85*; Nossin, *2.118*) or ancient (Popov, *2.127*) landslides in that they commonly have developed under different geomorphological and climatic conditions thousands or more years ago and cannot repeat themselves at present.

FORMING NAMES

The names applied to slope movements can be made progressively more informative, as more data are obtained, by building up a designation from several descriptor words, each of which has a defined meaning. For example, a slow, moist, translational debris slab slide means material moving along a planar surface of a little-disturbed mass of fragmented material having a D/L_c ratio of 0.1 or less, containing some water but none free, and moving at a rate between 1.5 m/month and 1.5 m/year (5 ft/month or year). Once all these particulars are established in the description, the movement could be referred to thereafter simply as a debris slide.

CAUSES OF SLIDING SLOPE MOVEMENTS

The processes involved in slides, as well as in other slope movements, comprise a continuous series of events from cause to effect. An engineer faced with a landslide is primarily interested in preventing the harmful effects of the slide. In many instances the principal cause of the slide cannot be removed, so it may be more economical to alleviate the effects continually or intermittently without attempting to remove the cause. Some slides occur in a unique environment and may last only a few seconds. The damage can be repaired, and the cause may be of only academic interest unless legal actions are to be taken. More often, however, landslides take place under the influence of geologic, topographic, or climatic factors that are common to large areas. The causes must then be understood if other similar slides are to be avoided or controlled.

Seldom, if ever, can a landslide be attributed to a single definite cause. As clearly shown by Zolotarev (*2.196*), the process leading to the development of the slide has its beginning with the formation of the rock itself, when its basic physical properties are determined, and includes all the subsequent events of crustal movements, erosion, and weathering. Finally, some action, perhaps trivial, sets a mass of material in motion downhill. The last action cannot be regarded as the only cause, even though it was necessary in the chain of events. As Sowers and Sowers (*2.161*, p. 506) point out,

> *In most cases a number of causes exist simultaneously and so attempting to decide which one finally produced failure is not only difficult but also incorrect. Often the final factor is nothing more than a trigger that set in motion an earth mass that was already on the verge of failure. Calling the final factor the cause is like calling the match that lit the fuse that detonated the dynamite that destroyed the building the cause of the disaster.*

In this connection, however, the determination of all the geologic causes of a landslide should not be confused with determination of legal responsibility. The interrelations of landslide causes are lucidly and graphically presented by Terzaghi (*2.175*). His work, that of Sharpe (*2.146*), Ladd (*2.92*), and Bendel (*2.6*), and that of more recent researchers, such as Záruba and Mencl (*2.193*), Skempton and Hutchinson (*2.154*), Krinitzsky and Kolb (*2.90*), Rapp (*2.131*), and Legget (*2.96*) were used in the preparation of this section.

All slides involve the failure of earth materials under shear stress. The initiation of the process can therefore be reviewed according to (a) the factors that contribute to increased shear stress and (b) the factors that contribute to low or reduced shear strength. Although a single action, such as addition of water to a slope, may contribute to both an increase in stress and a decrease in strength, it is helpful to separate the various physical results of such an action. The principal factors contributing to the sliding of slope-forming materials are outlined in the following discussion. The operation of many factors is self-evident and needs no lengthy description; some factors are only discussed briefly, or reference is made to literature that gives examples or treats the subject in detail.

Factors That Contribute to Increased Shear Stress

Removal of Lateral Support

The removal of lateral support is the commonest of all factors leading to instability, and it includes the following actions:

1. Erosion by (a) streams and rivers, which produce most natural slopes that are subject to sliding (Hutchinson, *2.67*; Jones, Embody, and Peterson, *2.79*; Eyles, *2.43*; Fleming, Spencer, and Banks, *2.48*; California Division of

Highways, *2.19*), (b) glaciers, which have deeply cut and oversteepened many valleys in mountainous regions that have been the sites of large slides and debris flows (Plafker, Ericksen, and Fernandez Concha, *2.126*), (c) waves and longshore or tidal currents (Wood, *2.189*; Ward, *2.186*; Hutchinson, *2.71*; Koppejan, Van Wamelon, and Weinberg, *2.89*), and (d) subaerial weathering, wetting and drying, and frost action;

2. Previous rock fall, slide (Kenney and Drury, *2.81*), subsidence, or large-scale faulting that create new slopes; and

3. Work of human agencies in which (a) cuts, quarries, pits, and canals (Van Rensburg, *2.181*; Piteau, *2.124*; Patton, *2.121*; Cording, *2.26*) are established, (b) retaining walls and sheet piling are removed, and (c) lakes and reservoirs are created and their levels altered (Müller, *2.112*; Jones, Embody, and Peterson, *2.79*; Lane, *2.93*; Dupree and Taucher, *2.39*).

Surcharge

Surcharge also results from both natural and human agencies. The surcharge from natural agencies may be

1. Weight of rain, hail, snow, and water from springs;
2. Accumulation of talus overriding landslide materials;
3. Collapse of accumulated volcanic material, producing avalanches and debris flows (Francis and others, *2.50*);
4. Vegetation (Gray, *2.53*; Pain, *2.120*); and
5. Seepage pressures of percolating water.

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The surcharge from human agencies may be

1. Construction of fill;
2. Stockpiles of ore or rock;
3. Waste piles (Bishop, *2.8*; Davies, *2.35*; Smalley, *2.156*);
4. Weight of buildings and other structures and trains; and
5. Weight of water from leaking pipelines, sewers, canals, and reservoirs.

Transitory Earth Stresses

Earthquakes have triggered a great many landslides, both small and extremely large and disastrous. Their action is complex, involving both an increase in shear stress (horizontal accelerations may greatly modify the state of stress within slope-forming materials) and, in some instances, a decrease in shear strength (Seed, *2.143*; Morton, *2.110*; Solonenko, *2.160*; Lawson, *2.95*; Hansen, *2.57*; Newmark, *2.117*; Simonett, *2.151*; Hadley, *2.55*; Gubin, *2.54*). Vibrations from blasting, machinery, traffic, thunder, and adjacent slope failures also produce transitory earth stresses.

Regional Tilting

A progressive increase in the slope angle through regional tilting is suspected as contributing to some landslides (Terzaghi, *2.175*). The slope must obviously be on the point of failure for such a small and slow-acting change to be effective.

Removal of Underlying Support

Examples of removal of underlying support include

1. Undercutting of banks by rivers (California Division of Highways, *2.19*) and by waves;
2. Subaerial weathering, wetting and drying, and frost action;
3. Subterranean erosion in which soluble material, such as carbonates, salt, or gypsum is removed and granular material beneath firmer material is worked out (Ward, *2.186*; Terzaghi, *2.174*);
4. Mining and similar actions by human agencies;
5. Loss of strength or failure in underlying material; and
6. Squeezing out of underlying plastic material (Záruba and Mencl, *2.193*, pp. 68-78).

Lateral Pressure

Lateral pressure may be caused by

1. Water in cracks and caverns,
2. Freezing of water in cracks,
3. Swelling as a result of hydration of clay or anhydrite, and
4. Mobilization of residual stress (Bjerrum, *2.9*; Krinitzsky and Kolb, *2.90*).

Volcanic Processes

Stress patterns in volcanic edifices and crater walls are modified by general dilation due to inflation or deflation of magma chambers, fluctuation in lava-lake levels, and increase in harmonic tremors (Tilling, Koyanagi, and Holcomb, *2.178*; Moore and Krivoy, *2.109*; Fiske and Jackson, *2.47*).

Factors That Contribute to Low or Reduced Shear Strength

The factors that contribute to low or reduced shear strength of rock or soil may be divided into two groups. The first group includes factors stemming from the initial state or inherent characteristics of the material. They are part of the geologic setting that may be favorable to landslides, exhibit little or no change during the useful life of a structure, and may exist for a long period of time without failure. The second group includes the changing or variable factors that tend to lower the shear strength of the material.

Initial State

Factors in the initial state of the material that cause low shear strength are composition, texture, and gross structure and slope geometry.

Composition

Materials are inherently weak or may become weak upon change in water content or other changes. Included especially are organic materials, sedimentary clays and shales, decomposed rocks, rocks of volcanic tuff that may weather to clayey material, and materials composed dominantly of soft platy minerals, such as mica, schist, talc, or serpentine.

Texture

The texture is a loose structure of individual particles in sensitive materials, such as clays, marl, loess, sands of low density, and porous organic matter (Aitchison, *2.2*; Bjerrum and Kenney, *2.11*; Cabrera and Smalley, *2.17*). Roundness of grain influences strength as compressibility and internal friction increase with angularity.

Gross Structure and Slope Geometry

Included in gross structure and slope geometry are

1. Discontinuities, such as faults, bedding planes, foliation in schist, cleavage, joints, slickensides, and brecciated zones (Skempton and Petley, *2.155*; Fookes and Wilson, *2.49*; Komarnitskii, *2.88*; St. John, Sowers, and Weaver, *2.138*; Van Rensburg, *2.181*; Jennings and Robertson, *2.74*; Bjerrum and Jørstad, *2.10*);
2. Massive beds over weak or plastic materials (Záruba and Mencl, *2.193*; Nemčok, *2.113*);
3. Strata inclined toward free face;
4. Alternation of permeable beds, such as sand or sandstone, and weak impermeable beds, such as clay or shale (Henkel, *2.59*); and
5. Slope orientation (Rice, Corbett, and Bailey, *2.134*; Shroder, *2.150*).

Changes Due to Weathering and Other Physicochemical Reactions

The following changes can occur because of weathering and other physicochemical reactions:

1. Softening of fissured clays (Skempton, *2.153*; Sangrey and Paul, *2.139*; Eden and Mitchell, *2.41*);
2. Physical disintegration of granular rocks, such as granite or sandstone, under action of frost or by thermal expansion (Rapp, *2.131*);
3. Hydration of clay minerals in which (a) water is absorbed by clay minerals and high water contents decrease cohesion of all clayey soils, (b) montmorillonitic clays swell and lose cohesion, and (c) loess markedly consolidates upon saturation because of destruction of the clay bond between silt particles;
4. Base exchange in clays, i.e., influence of exchangeable ions on physical properties of clays (Sangrey and Paul, *2.139*; Liebling and Kerr, *2.99*; Torrance, *2.179*);
5. Migration of water to weathering front under electrical potential (Veder, *2.184*);
6. Drying of clays that results in cracks and loss of cohesion and allows water to seep in;
7. Drying of shales that creates cracks on bedding and shear planes and reduces shale to chips, granules, or smaller particles; and
8. Removal of cement by solution.

Changes in Intergranular Forces Due to Water Content and Pressure in Pores and Fractures

Buoyancy in saturated state decreases effective intergranular pressure and friction. Intergranular pressure due to capillary tension in moist soil is destroyed upon saturation. Simple softening due to water and suffusion and slaking are discussed by Mamulea (*2.101*).

Changes can occur because of natural actions, such as rainfall and snowmelt, and because of a host of human activities, such as diversion of streams, blockage of drainage, irrigation and ponding, and clearing of vegetation and deforestation.

Crozier (*2.30*, *2.31*), Shroder (*2.150*), and Špůrek (*2.163*) discuss the general effect of climate; Temple and Rapp (*2.169*) Williams and Guy (*2.188*), Jones (*2.78*), and So (*2.158*), catastrophic rainfall; Conway (*2.25*), Denness (*2.38*), and Piteau (*2.124*), effect of groundwater; Gray (*2.53*), Bailey (*2.4*), Cleveland (*2.22*), Rice, Corbett, and Bailey (*2.134*), and Swanston (*2.165*), deforestation; Peck (*2.123*) and Hirao and Okubo (*2.60*), correlation of rainfall and movement; and Shreve (*2.148*), Voight (*2.185*), Kent (*2.82*), and Goguel and Pachoud (*2.52*), gaseous entrainment or cushion.

Changes in Structure

Changes in structure may be caused by fissuring of shales and preconsolidated clays and fracturing and loosening of rock slopes due to release of vertical or lateral restraints in valley walls or cuts (Bjerrum, *2.9*; Aisenstein, *2.1*; Ferguson, *2.45*; Matheson and Thomson, *2.102*; Mencl, *2.106*). Disturbance or remolding can affect the shear strength of materials composed of fine particles, such as loess, dry or saturated loose sand, and sensitive clays (Gubin, *2.54*; Youd, *2.191*; Smalley, *2.156*; Mitchell and Markell, *2.108*).

Miscellaneous Causes

Other causes of low shear strength are (a) weakening due to progressive creep (Suklje, *2.164*; Ter-Stepanian, *2.172*; Trollope, *2.180*; Piteau, *2.124*) and actions of tree roots (Feld, *2.44*) and burrowing animals.

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Soil Mechanics in Engineering Practice

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CONTENTS

Part II. Theoretical Soil Mechanics

Part III. Problems of Design and Construction

ART. 35 STABILITY OF SLOPES

Introduction

The failure of a mass of soil located beneath a slope is called a *slide*. It involves a downward and outward movement of the entire mass of soil that participates in the failure.

Slides may occur in almost every conceivable manner, slowly or suddenly, and with or without any apparent provocation. Usually, slides are due to excavation or to undercutting the foot of an existing slope. However, in some instances, they are caused by a gradual disintegration of the structure of the soil, starting at hair cracks which subdivide the soil into angular fragments. In others, they are caused by an increase of the porewater pressure in a few exceptionally permeable layers, or by a shock that liquefies the soil beneath the slope (Article 49). Because of the extraordinary variety of factors and processes that may lead to slides, the conditions for the stability of slopes usually defy theoretical analysis. Stability computations based on test results can be relied on only when the conditions specified in the different sections of this article are strictly satisfied. Moreover, it should always be remembered that various undetected discontinuities in the soil, such as systems of hair cracks, remnants of old surfaces of sliding, or thin seams of water-bearing sand, may completely invalidate the results of the computations 35

Slopes on Dry Cohesionless Sand

A slope underlain by clean dry sand is stable regardless of its height, provided the angle β between the slope and the horizontal is equal to or smaller than the angle of internal friction ϕ for the sand in a loose state. The factor of safety of the slope with respect to sliding may be expressed by the equation,

$$F = \frac{\tan \phi}{\tan \beta} \tag{35.1}$$

No slope on clean sand can exist with a slope angle greater than ϕ, irrespective of its height.

Since very few natural soils are perfectly cohesionless, the remainder of this article deals with slopes underlain by cohesive materials.

General Character of Slides in Homogeneous Cohesive Soil

A cohesive material having a shearing resistance

$$s = c + p \tan \phi$$

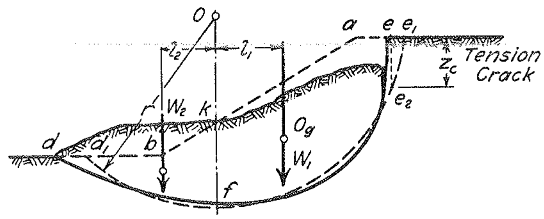

Fig. 35.1. Deformation associated with slope failure.

can stand with a vertical slope at least for a short time, provided the height of the slope is somewhat less than H_c (Eq. 28.11). If the height of a slope is greater than H_c, the slope is not stable unless the slope angle β is less than 90°. The greater the height of the slope, the smaller must be the angle β. If the height is very great compared to H_c, the slope will fail unless the slope angle β is equal to or less than ϕ.

The failure of a slope in a cohesive material is commonly preceded by the formation of tension cracks behind the upper edge of the slope, as shown in Fig. 35.1. The force which produces the tension cracks behind the edge of a vertical slope is represented by the triangle *ace* in Fig. 28.3*b*. Sooner or later, the opening of the cracks is followed by sliding along a curved surface, indicated by the full line in Fig. 35.1. Usually the radius of curvature of the surface of sliding is least at the upper end, greatest in the middle, and intermediate at the lower end. The curve, therefore, resembles the arc of an ellipse. If the failure occurs along a surface of sliding that intersects the slope at or above its toe (Fig. 35.2*a*), the slide is known as a *slope failure*. On the other hand, if the soil beneath the level of the toe of the slope is unable to sustain the weight of the overlying material, the failure occurs along a surface that passes at some distance below the toe of the slope. A failure of this type, shown in Fig. 35.2*b*, is known as a *base failure*.

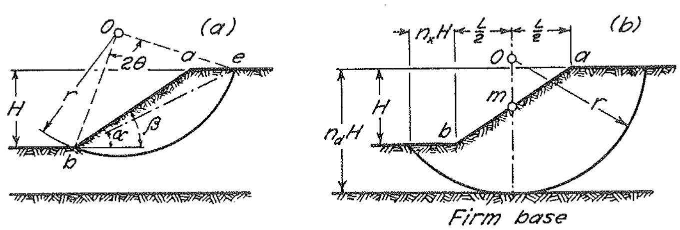

Fig. 35.2. Position of critical circle for (*a*) slope failure (after W. Fellenius 1927). (*b*) Base failure.

In stability computations the curve representing the real surface of sliding is usually replaced by an arc of a circle or of a logarithmic spiral. Either procedure is as legitimate as Coulomb's assumption of a plane surface of sliding in connection with retaining wall problems (Article 30). In the following discussions only the circle will be used as a substitute for the real surface of sliding.

Purpose of Stability Computations

In engineering practice, stability computations serve as a basis either for the redesign of slopes after a failure or for choosing slope angles in accordance with specified safety requirements in advance of construction.

Local failures on the slopes of cuts or fills are common during the construction period. They indicate that the average value of the minimum shearing resistance of the soil has been overestimated. Since such failures constitute large-scale shear tests, they offer excellent opportunities for evaluating the real minimum shearing resistance and for avoiding further accidents on the same job by changing the design in accordance with the findings. The general procedure is to determine the position of the surface of sliding by means of test borings, slope indicators, or shafts; to estimate the weights of the various parts of the sliding mass that tended to produce or to oppose the slide; and to compute the average shearing resistance s of the soil necessary to satisfy the conditions for equilibrium of the mass. 37

In order to design a slope in a region where no slides have occurred, the average shearing resistance s must be estimated or determined in advance of construction. Methods for evaluating the shearing resistance are discussed in Articles 17 and 18. After the value of s has been determined, the slope angle can be chosen on the basis of theory in such a manner that the slope satisfies the specified safety requirements. It is obvious that this method can be used only if the soil conditions permit a fairly reliable determination of s on the basis of the results of soil tests.

Computation of Shearing Resistance from Slide Data

The method for determining the average shearing resistance of soils on the basis of slide data is illustrated by Fig. 35.1. The depth z_c of the tension cracks and the shape of the surface of sliding are ascertained by field measurements. The line of sliding is then replaced by the arc of a circle having a radius r and its center at O. Equilibrium requires that

$$W_1l_1 = W_2l_2 + sr\,\overset{\frown}{d_1e_2}$$

from which

$$s = \frac{W_1 l_1 - W_2 l_2}{r\,\overset{\frown}{d_1 e_2}}$$

where W_1 is the weight of the slice *akfe* which tends to produce failure, and W_2 is the weight of slice kbd_1f which tends to resist it.

If the shape of the surface of sliding is such that it cannot be represented even approximately by an arc of a circle, the procedure must be modified according to the methods described subsequently in connection with composite surfaces of sliding.

Procedure for Investigating Stability of Slopes

In order to investigate whether or not a slope on soil with known shear characteristics will be stable, it is necessary to determine the diameter and position of the circle that represents the surface along which sliding will occur. This circle, known as the *critical circle,* must satisfy the requirement that the ratio between the shearing strength of the soil along the surface of sliding and the shearing force tending to produce the sliding must be a minimum. Hence, the investigation belongs to the category of maximum and minimum problems exemplified by Coulomb's theory (Article 30) and the theory of passive earth pressure (Article 32).

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After the diameter and position of the critical circle have been determined, the factor of safety F of the slope with respect to failure may be computed by means of the relation (Fig. 35.1)

$$F = \frac{sr\,\overset{\frown}{d_1 e_2}}{W_1 l_1 - W_2 l_2} \tag{35.2}$$

wherein r represents the radius of the critical circle and d_1e_2 the length of the surface of sliding.

Like the passive earth pressure of a mass of soil, the stability of a slope may be investigated by trial or, in simple cases, by analytical methods. To make the investigation by trial, different circles are selected, each representing a potential surface of sliding. For each circle, the value F (Eq. 35.2) is computed. The minimum value represents the factor of safety of the slope with respect to sliding, and the corresponding circle is the critical circle.

The analytical solutions can rarely be used to compute the factor of safety of a slope under actual conditions, because they are based on greatly simplified assumptions. They are valuable, however, as a guide for estimating the position of the center of the critical circle and for ascertaining the probable character of the failure. In addition, they may serve as a means for judging whether a given slope will be unquestionably safe, unquestionably unsafe, or of doubtful stability. If the stability appears doubtful, the factor of safety with respect to failure should be computed according to the procedure described in the preceding paragraph.

The analytical solutions are based on the following assumptions: Down to a given level below the toe of the slope, the soil is perfectly homogeneous. At this level, the soil rests on the horizontal surface of a stiffer stratum, known as the *firm base*, which is not penetrated by the surface of sliding. The slope is considered to be a plane, and it is located between two horizontal plane surfaces, as shown in Fig. 35.2. Finally, the weakening effect of tension cracks is disregarded, because it is more than compensated by the customary margin of safety. The following paragraphs contain a summary of the results of the investigations.

Slopes on Soft Clay

The average shearing resistance s per unit of area of a potential surface of sliding in homogeneous clay under undrained ($\phi = 0$) conditions (Article 18) is roughly equal to one-half the unconfined compressive strength q_u of the clay. This value of s is referred to briefly as the cohesion c. That is,

$$s = \tfrac{1}{2}q_u = c \qquad (18.5)$$

If c is known, the critical height H_c of a slope having a given slope angle β can be expressed by the equation,

$$H_c = N_s \frac{c}{\gamma} \qquad (35.3)$$

In this equation the *stability factor* N_s is a pure number. Its value depends only on the slope angle β and on the *depth factor* n_d (Fig. 35.2*b*) which expresses the depth at which the clay rests on a firm base. If a slope failure occurs, the critical circle is usually a *toe circle* that passes through the toe b of the slope (Fig. 35.2*a*). However, if the firm base is located at a short distance below the level of b, the critical circle may be a *slope circle* that is tangent to the firm base and that intersects the slope above the toe b. This type of failure is not shown in Fig. 35.2. If a base failure occurs, the critical circle is known as a *midpoint circle*, because its center is located on a vertical line through the midpoint m of the slope (Fig. 35.2*b*). The midpoint circle is tangent to the firm base.

The position of the critical circle with reference to a given slope depends on the slope angle β and the depth factor n_d. Figure 35.3 contains a summary of the results of pertinent theoretical investigations. According to this figure, the failure of all slopes rising at an angle of more than 53° occurs along a toe circle. If β is smaller than 53°, the type of failure depends on the value of the depth factor n_d and, at low values of n_d, also on the slope angle β. If n_d is equal to 1.0, failure occurs along a slope circle. If n_d is greater than about 4.0, the slope fails along a midpoint circle tangent to the firm base, regardless of the value of β. If n_d is intermediate in value between 1.0 and 4.0, failure occurs along a slope circle if the point representing the values of n_d and β lies above the shaded area in Fig. 35.3. If the point lies within the shaded area, failure occurs along a toe circle. If the point is below the shaded area, the slope fails along a midpoint circle tangent to the firm base.

If the slope angle β and the depth factor n_d are given, the value of the corresponding stability factor N_s (Eq. 35.3) can be obtained without computation from Fig. 35.3. The value of N_s determines the critical height H_c of the slope.

If failure occurs along a toe circle, the center of the critical circle can be located by laying off the angles α and 2θ, as shown in Fig. 35.2*a*.

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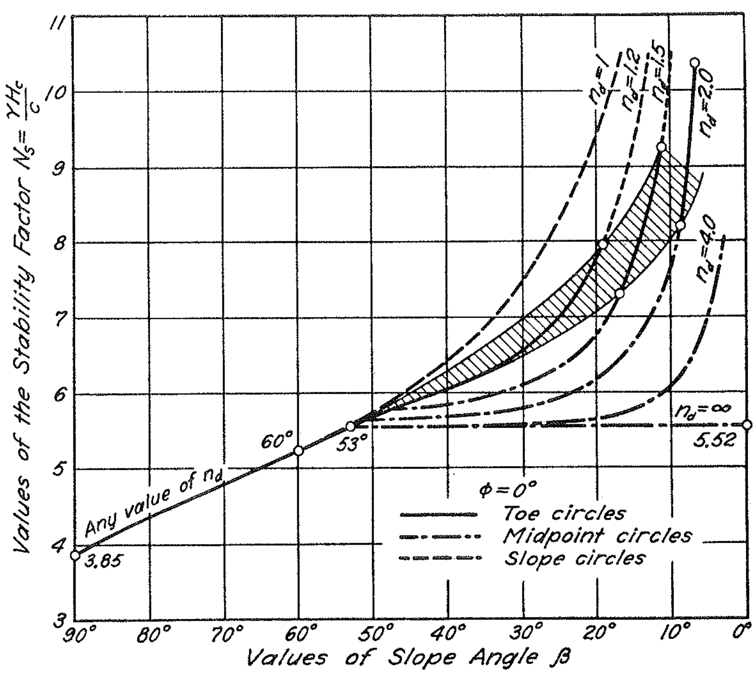


Fig. 35.3. Relation for frictionless material between slope angle β and stability factor N_s for different values of depth factor n_d (after Taylor 1937).

Values of α and θ for different slope angles β are given in Fig. 35.4*a*. If failure occurs along a midpoint circle tangent to the firm base, the position of the critical circle is determined by the horizontal distance n_xH from the toe of the slope to the circle (Fig. 35.2*b*). Values of n_x can be estimated for different values of n_d and β by means of the chart (Fig. 35.4*b*).

If the clay beneath a slope consists of several layers with different average cohesion c_1, c_2, etc., or if the surface of the ground is irregular (Fig. 35.5), the center of the critical circle must be determined by trial and error. It is obvious that the longest part of the real surface of sliding will be located within the softest stratum. Therefore, the trial circle should also satisfy this condition. If one of the upper layers is relatively soft, the presence of a firm base at considerable depth may not

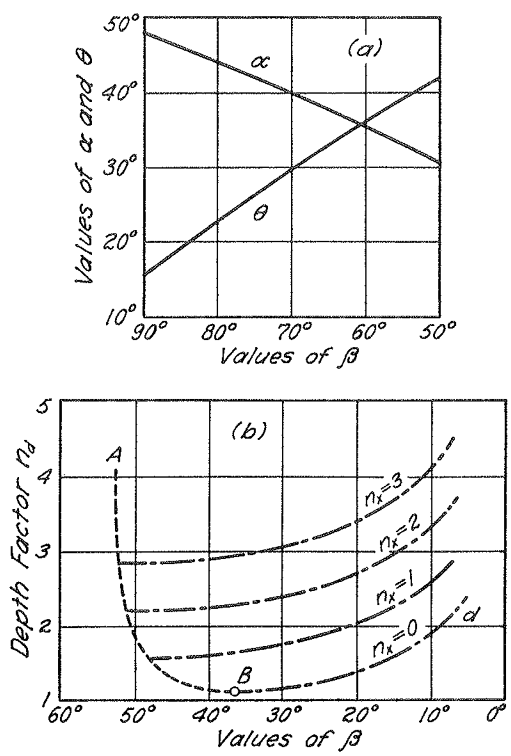


Fig. 35.4. (*a*) Relation between slope angle β and parameters α and θ for location of critical toe circle when β is greater than 53°. (*b*) Relation between slope angle β and depth factor n_d for various values of parameter n_x (after W. Fellenius 1927).

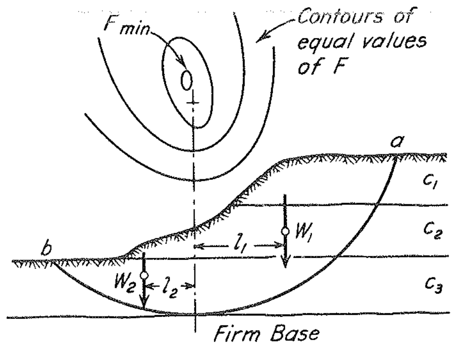


Fig. 35.5. Base failure in stratified cohesive soil.

enter into the problem, because the deepest part of the surface of sliding is likely to be located entirely within the softest stratum. For example, if the cohesion c_2 of the second stratum in Fig. 35.5 is much smaller than the cohesion c_3 of the underlying third layer, the critical circle will be tangent to the upper surface of the third stratum instead of the firm base.

For each trial circle we compute the average shearing stress t which
42 must act along the surface of sliding to balance the difference between the moment W_1l_1 of the driving weight and the resisting moment W_2l_2. The value of t is

$$t = \frac{W_1l - W_2l_2}{r\,\widehat{ab}}$$

Then, on the basis of the known values of c_1, c_2, c_3, etc., we compute the average value of the cohesion c of the soil along the sliding surface. The factor of safety of the slope against sliding along the circular trial surface is

$$F = \frac{c}{t} \tag{35.4}$$

The value of F is inscribed at the center of the circle. After values of F have been determined for several trial circles, curves of equal values of F are plotted (Fig. 35.5). These curves may be considered as contour lines of a depression. The center of the critical circle is located at the bottom of the depression. The corresponding value $F_{\min}$ is the factor of safety of the slope with respect to sliding.

If it is not obvious which of two layers may constitute the firm base for the critical circle, trial circles must be investigated separately for each possibility and the corresponding values of $F_{\min}$ determined. The smaller of the two values is associated with the firm base that governs the failure and is the factor of safety of the slope.

Slopes on Soils with Cohesion and Internal Friction

If the shearing resistance of the soil can be expressed approximately by the equation

$$s = c + p \tan \phi$$

the stability of slopes on the soil can be investigated by the procedure illustrated by Fig. 35.6*a*. The forces acting on the sliding mass are its weight W, the resultant cohesion C, and the resultant F of the normal and frictional forces acting along the surface of sliding. The resultant cohesion C acts in a direction parallel to the chord *de* and is equal to the unit cohesion c multiplied by the length L of the chord. The distance x from the center of rotation to C is determined by the condition that

$$Cx = cLx = c\,\overset{\frown}{de}\,r$$

whence $x = \overset{\frown}{de}\,r/L$. Therefore, the force C is known. The weight W is also known. Since the forces C, W, and F are in equilibrium, the force F must pass through the point of intersection of W and C. Hence, the magnitude and line of action of F can be determined by constructing the polygon of forces. 43

If the factor of safety against sliding is equal to unity, the slope is on the verge of failure. Under this condition each of the elementary reactions dF in Fig. 35.6*a* must be inclined at the angle ϕ to the normal to the circle of sliding. As a consequence, the line of action of each elementary reaction is tangent to a circle, known as the *friction circle,* having a radius

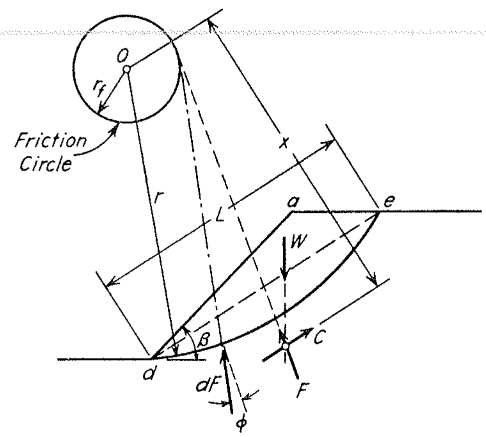
$$r_f = r \sin \phi$$

and having its center at the center of the circle of sliding. The line of action of the resultant reaction F is tangent to a circle having a radius slightly greater than r_f, but as a convenient approximation we assume that at a factor of safety equal to unity the line of action of F is also tangent to the friction circle. The corresponding error is small and is on the safe side.

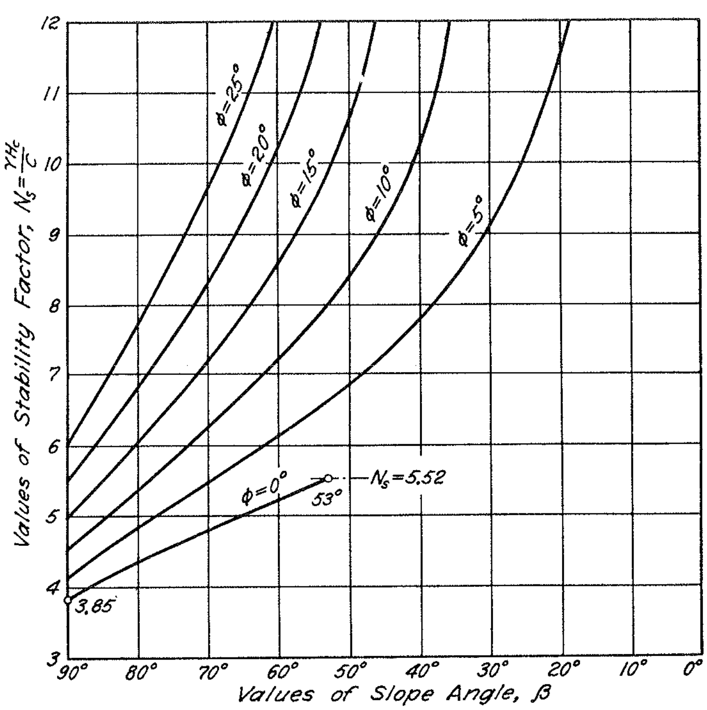
For a given value of ϕ the critical height of a slope which fails along a toe circle is given by the equation,

$$H_c = N_s \frac{c}{\gamma}$$

which is identical with Eq. 35.3, except that N_s depends not only on β but also on ϕ. Figure 35.6*b* shows the relationship between β and N_s



(a)



(b)

Fig. 35.6. Failure of slope in material having cohesion and friction. (*a*) Diagram illustrating friction-circle method. (*b*) Relation between slope angle β and stability factor N_s for various values of ϕ (after Taylor 1937).

for different values of ϕ. At a given value of the slope angle β, N_s increases at first slowly and then more rapidly with increasing values of ϕ. When $\phi = \beta$, N_s becomes infinite.

All the points on the curves shown in Fig. 35.6*b* correspond to failures along toe circles, because theory has shown that the possibility of a base failure does not exist unless ϕ is smaller than approximately 3°. Therefore, if a typical base failure has occurred in a fairly homogeneous soil in the field, it can be concluded that with respect to total stresses the value of ϕ for the soil at the time of the slide was close to zero.

Irregular Slopes on Nonuniform Soils

If a slope has an irregular surface that cannot be represented by a straight line, or if the surface of sliding is likely to pass through several materials with different values of c and ϕ, the stability can be investigated conveniently by the *method of slices.* According to this procedure a trial circle is selected (Fig. 35.7*a*) and the sliding mass subdivided into a number of vertical slices 1, 2, 3, etc. Each slice, such as slice 2 shown in Fig. 35.7*b*, is acted upon by its weight W, by shear forces T and normal forces E on its sides, and by a set of forces on its base. These include the shearing force S and the normal force P. The forces on each slice, as well as those acting on the sliding mass as a whole, must satisfy the conditions of equilibrium. However, the forces T and E depend on the deformation and the stress-strain characteristics of the slide material and cannot be evaluated rigorously. They can be approximated with sufficient accuracy for practical purposes.

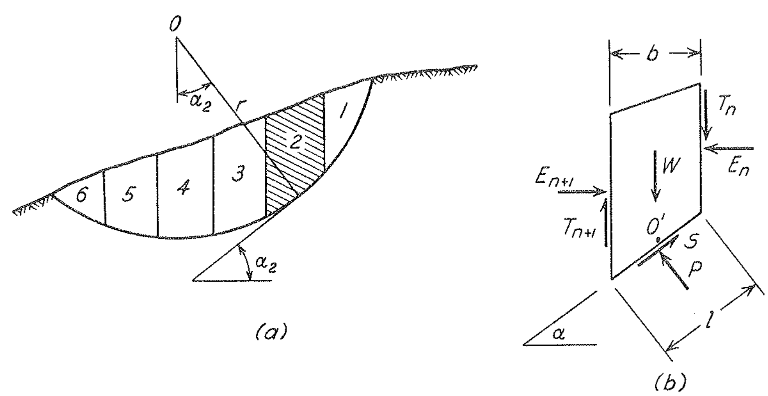


Fig. 35.7. Method of slices for investigating equilibrium of slope located above water table. (*a*) Geometry pertaining to one circular surface of sliding. (*b*) Forces on typical slice such as slice 2 in (*a*).

The simplest approximation consists of setting these forces equal to zero. Under these circumstances, if the entire trial circle is located above the water table and there are no excess pore pressures, equilibrium of the entire sliding mass requires that

$$r\Sigma W \sin \alpha = r\Sigma S \tag{35.5}$$

If s is the shearing strength of the soil along l, then

$$S = \frac{s}{F} l = \frac{s}{F} \frac{b}{\cos \alpha} \tag{35.6}$$

and

$$r \sum W \sin \alpha = \frac{r}{F} \sum \frac{sb}{\cos \alpha} \tag{35.7}$$

whence

$$F = \frac{\Sigma(sb/\cos \alpha)}{\Sigma W \sin \alpha} \tag{35.8}$$

The shearing strength s, however, is determined by

$$s = c + p \tan \phi$$

46 where p is the normal stress across the surface of sliding l. To evaluate p we consider the vertical equilibrium of the slice (Fig. 35.7*b*), whence

$$W = S \sin \alpha + P \cos \alpha$$

and

$$p = \frac{P}{l} = \frac{P \cos \alpha}{b} = \frac{W}{b} - \frac{S}{b} \sin \alpha \tag{35.9}$$

Therefore

$$s = c + \left(\frac{W}{b} - \frac{S}{b} \sin \alpha\right) \tan \phi = c + \left(\frac{W}{b} - \frac{s}{F} \tan \alpha\right) \tan \phi$$

and

$$s = \frac{c + (W/b) \tan \phi}{1 + (\tan \alpha \tan \phi)/F} \tag{35.10}$$

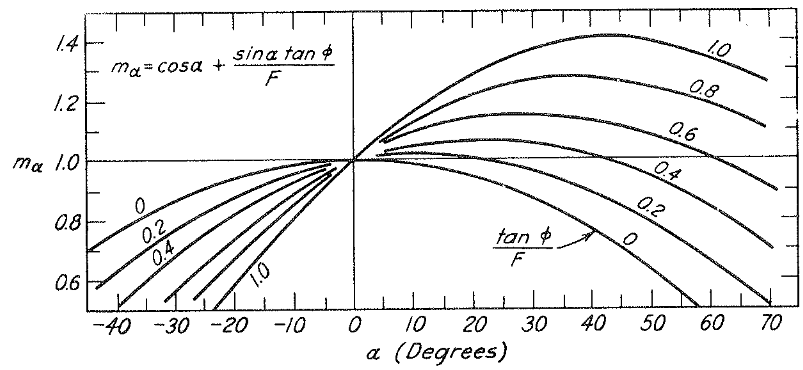
Let

$$m_\alpha = \left(1 + \frac{\tan \alpha \tan \phi}{F}\right) \cos \alpha \tag{35.11}$$

Then

$$F = \frac{\displaystyle\sum \frac{[c + (W/b) \tan \phi]b}{m_\alpha}}{\Sigma W \sin \alpha} \tag{35.12}$$

Equation 35.12, which gives the factor of safety F for the trial circle under investigation, contains on the right-hand side the quantity m_α (Eq. 35.11) which is itself a function of F. Therefore, Eq. 35.12 must be solved by successive approximations in which a value of $F = F_1$ is assumed and used for calculation of m_α, whereupon F is then computed. If the value of F differs significantly from F_1, the calculation is repeated. Convergence is very rapid. The calculations are facilitated by the chart (Fig. 35.8*a*) from which values of m_α can be taken (Janbu et al. 1956), and by a tabular arrangement of the computations (Fig. 35.8*b*).



(*a*)

Values from cross section								
	1	2	3	4	5	6	7	8
Slice No.	$\alpha°$	$\sin\alpha$	W	$W\sin\alpha$	$c + \frac{W}{b}\tan\phi$	$(5)\cdot b$	m_α $F_a =$	(6)/(7)
								$\Sigma(8)$

$$\text{For first trial, } F_a = \frac{\Sigma(6)}{\Sigma(4)} \qquad F = \frac{\Sigma(8)}{\Sigma(4)}$$

(*b*)

Fig. 35.8. Calculation of factor of safety for slope if surface of sliding is circular and forces between slices are neglected. (*a*) Chart for evaluating factor m_α. (*b*) Tabular form for computation.

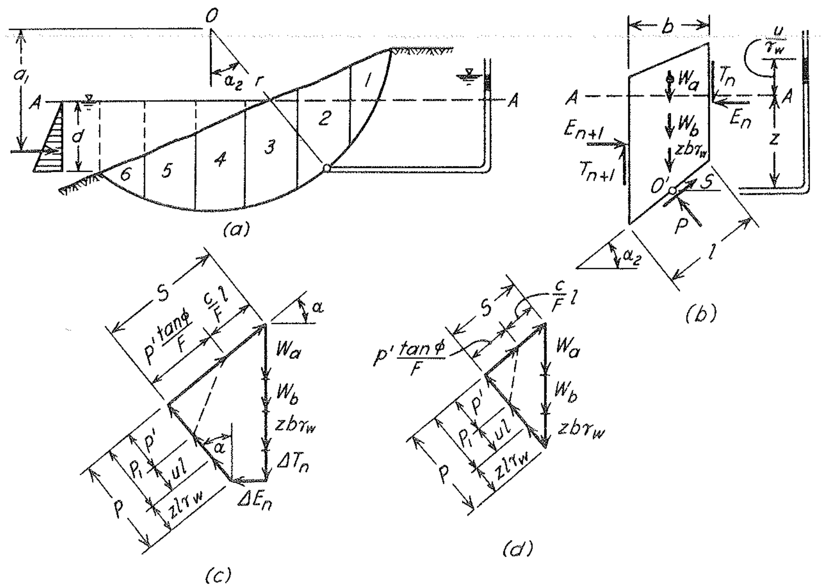


Fig. 35.9. Method of slices for circular surface of sliding if slope is partly submerged. (*a*) Geometry pertaining to one surface of sliding. (*b*) Forces acting on typical slice, such as slice 2 in (*a*). (*c*) Force polygon for slice 2 if all forces are considered. (*d*) Force polygon for slice 2 if forces *T* and *E* on sides of slice are considered to be zero.

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Inasmuch as the calculations outlined in Fig. 35.8 refer to only one trial circle, they must be repeated for other circles until the minimum value of F is found.

In general, the slope may be partly submerged and there will be pore pressures acting along the trial circle (Fig. 35.9*a*). The magnitudes of the pore pressures depend upon the conditions of the problem. In some instances they may be estimated by means of a flow net (Article 23), by means of soil tests, or on the basis of field observations. If the level of the external water surface is denoted by $A - A$, the weight W of the slice (Fig. 35.9*b*) may be written as

$$W = W_a + W_b + zb\gamma_w \tag{35.13}$$

where W_a is the weight of that part of the slice above $A - A$, W_b is the submerged weight of the part below $A - A$, and $zb\gamma_w$ is the weight of a volume of water equal to the submerged portion of the slice. If the entire slice is located beneath water level, as slice 5 (Fig. 35.9*a*), the weight of the water above the slice must be included in $zb\gamma_w$. The pore pressure at the midpoint $0'$ of the base of the slice is $z\gamma_w + u$, where u is the excess pore pressure with respect to the external water level. If the external water level A-A is located below O' on the base of the slice (Fig. 35.9*b*), the pore pressure at O' is h/γ_w, where h is the height to which the water would rise in a piezometer at O'. If the pore pressure is due to capillarity, h is negative.

Since the forces acting on the slice are in equilibrium, they may be represented by the force polygon (Fig. 35.9*c*). The normal force P consists of an effective component P', the force ul caused by the excess pore pressure, and the force $zl\gamma_w$ caused by the hydrostatic pressure of the water with respect to $A - A$. The shearing stress t along the surface of sliding is

$$t = \frac{s}{F} = \frac{1}{F}(c + \bar{p}\tan\phi) = \frac{1}{F}\left[c + \left(\frac{P}{l} - z\gamma_w - u\right)\tan\phi\right] \quad (35.14)$$

whence

$$S = t \cdot l = \frac{1}{F}[cl + (P - zl\gamma_w - ul)\tan\phi] = \frac{1}{F}(cl + P'\tan\phi) \quad (35.15)$$

Equilibrium of the entire slide with respect to moments about the center of the trial circle requires that 49

$$\sum (W_a + W_b + zb\gamma_w)r\sin\alpha = \sum S \cdot r + \tfrac{1}{2}\gamma_w d^2 a_1$$
$$= \frac{1}{F}\sum (cl + P'\tan\phi)r + \tfrac{1}{2}\gamma_w d^2 a_1 \quad (35.16)$$

However, the water below level $A - A$ is in equilibrium, whence

$$\Sigma zb\gamma_w r \sin\alpha = \tfrac{1}{2}\gamma_w d^2 a_1 \quad (35.17)$$

Therefore,

$$\sum (W_a + W_b)r\sin\alpha = \frac{1}{F}\sum (cl + P'\tan\phi)r \quad (35.18)$$

and

$$F = \frac{\Sigma(cl + P'\tan\phi)}{\Sigma(W_a + W_b)\sin\alpha} \quad (35.19)$$

The value of F (Eq. 35.19) depends upon P' which may be determined for each slice from the force polygon (Fig. 35.9*c*). If the surface of sliding is circular, the influence of the forces T and E between the slices is relatively small and P' can usually be evaluated with sufficient accuracy on the assumption that the forces T and E are equal to zero. The force polygon then reduces to Fig. 35.9*d*, whence

$$W_a + W_b + zb\gamma_w = (zl\gamma_w + P' + ul)\cos\alpha + \left(P'\frac{\tan\phi}{F} + \frac{cl}{F}\right)\sin\alpha \tag{35.20}$$

and

$$P' = \frac{W_a + W_b - ub - \dfrac{cl}{F}\sin\alpha}{m_\alpha} \tag{35.21}$$

Substitution of Eq. 35.21 into 35.19 gives

$$F = \frac{\displaystyle\sum \frac{[cb + (W_a + W_b - ub)\tan\phi]}{m_\alpha}}{\Sigma(W_a + W_b)\sin\alpha} \tag{35.22}$$

Equation 35.22, like Eq. 35.12, must be solved by successive approximations because the factor of safety F is contained in m_α which appears on the right-hand side. It may be noted that the influence of the external water level is fully taken into account by the use of the submerged weight W_b, and that the excess pore pressure u is calculated for
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the base of each slice as explained in connection with Eq. 35.13.

The procedure described in the preceding paragraphs may be modified to take into account the forces T and E between the slices (Bishop 1955, Janbu 1954*a*). If the surface of sliding is circular, however, the improvement in accuracy is not likely to exceed 10 to 15% and the additional effort is not usually justified. On the other hand, if the surface of sliding is not circular the error may be significant. These circumstances will be considered in the next section. The procedures that will be developed may, if desired, be used to take into account the forces between slices for a circular surface of sliding as well.

Composite Surface of Sliding

In many instances the geometric or geologic conditions of the problem are such that the surface of sliding may not be even approximately circular. For these conditions, the method of slices can be extended (Janbu 1954*a*, Nonveiller 1965).

A sliding mass with a noncircular surface of sliding is shown in Fig. 35.10. The forces acting on any slice n are represented in the same manner as those shown in Fig. 35.9*b*, and the polygon of forces is identical to that in Fig. 35.9*c*.

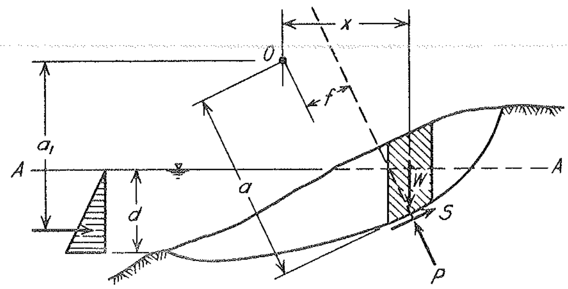

Fig. 35.10. Geometry of method of slices for investigating equilibrium of slope if surface of sliding is not circular.

The equilibrium of the entire sliding mass with respect to moments about the arbitrary pole O requires that

$$\Sigma Wx = \Sigma(Sa + Pf) + \tfrac{1}{2}\gamma_w d^2 a_1 \qquad (35.23)$$

whence, from Eq. 35.15

$$\sum (W_a + W_b + zb\gamma_w)x = \frac{1}{F}\sum (cl + P' \tan\phi)a + \sum Pf + \tfrac{1}{2}\gamma_w d^2 a_1$$

and

$$F = \frac{\Sigma(cl + P' \tan\phi)a}{\Sigma(W_a + W_b + zb\gamma_w)x - \Sigma Pf - \tfrac{1}{2}\gamma_w d^2 a_1} \qquad (35.24)$$

However, the water below level $A - A$ is in equilibrium, whence

$$\Sigma zb\gamma_w x - \tfrac{1}{2}\gamma_w d^2 a_1 = \Sigma zl\gamma_w f = \Sigma(P - P_1)f \qquad (35.25)$$

where

$$P_1 = P - zl\gamma_w$$

Equation 35.24 then becomes

$$F = \frac{\Sigma(cl + P' \tan\phi)a}{\Sigma(W_a + W_b)x - \Sigma P_1 f} \qquad (35.26)$$

This expression can be evaluated if P' and P_1 are known. These quantities may be determined from the force polygon (Fig. 35.9*c*). Summation of vertical components leads to

$$W_a + W_b + \Delta T_n + zb\gamma_w = zl\gamma_w \cos\alpha + (P' + ul)\cos\alpha + \frac{1}{F}(cl + P' \tan\phi)\sin\alpha$$

whence

$$P' = \frac{W_a + W_b + \Delta T_n - ub - (c/F)b \tan \alpha}{m_\alpha} \tag{35.27}$$

Moreover,

$$P_1 = P' + ul = \frac{W_a + W_b + \Delta T_n + (1/F)(ub \tan \phi - cb) \tan \alpha}{m_\alpha} \tag{35.28}$$

By substituting Eqs. 35.27 and 35.28 in 35.26 and combining terms, we find

$$F = \frac{\Sigma[cb + (W_a + W_b + \Delta T_n - ub) \tan \phi](a/m_\alpha)}{\sum (W_a + W_b)x - \sum \left[W_a + W_b + \Delta T_n + (ub \tan \phi - cb) \dfrac{\tan \alpha}{F} \right] (f/m_\alpha)} \tag{35.29}$$

This equation must be solved by successive approximations because the factor of safety F occurs on the right-hand side explicitly as well as in the quantity m_α. Furthermore, the value of F depends on ΔT_n. As a first approximation, ΔT_n may be set equal to zero. The calculations are facilitated by the chart (Fig. 35.8*a*) and a tabular arrangement (Fig. 35.11). Inasmuch as the value of F determined in this manner refers to only one trial surface, the calculations must be repeated for other surfaces until the minimum value of F is found.

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For most practical problems involving a noncircular surface of sliding, the assumption that ΔT_n is equal to zero leads to sufficiently accurate results. If the cross section of the surface of sliding departs significantly from a circular shape, the use of Eq. 35.29 with $\Delta T_n = 0$ is preferable to the assumption of a circular cross section and the use of Eq. 35.22. However, if greater refinement is justified, values of ΔT_n may be inserted in Eq. 35.29 and the factor of safety recalculated. The calculations are laborious.

If the values of T and E are not zero, they must satisfy the conditions for equilibrium of the entire sliding mass in vertical and horizontal directions. That is

$$\Sigma \Delta T_n = 0 \tag{35.30}$$

$$\Sigma \Delta E_n + \tfrac{1}{2}\gamma_w d^2 = 0 \tag{35.31}$$

Furthermore, for each slice, ΔT_n and ΔE_n are related in accordance with the requirements of the force polygon (Fig. 35.9*c*). By resolving the forces in the direction of S, we obtain

$$S = \Delta E_n \cos \alpha + (W_a + W_b + \Delta T_n + zb\gamma_w) \sin \alpha$$

Col.	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
Slice	b	a	x	f	α	$\tan\alpha$	c	$\tan\phi$	cb	ub	W_a	W_b	ΔT_n	$W_a + W_b + \Delta T_n$	(14) $-$ ub	(15) $\tan\phi$	(9) + (16)
1																	
2																	
:																	
n																	

Col.	18	19	20	21	22	23	24	25	26	27	28	29	30
Slice	(17) $\cdot a$	$W_a + W_b$	(19) $\cdot x$	$ub\tan\phi$	(21) $-$ cb	(22) $\tan\alpha$	F_1	$\frac{(23)}{F_1}$	(14) + (25)	(26) $\cdot f$	m_α	$\frac{(18)}{m_\alpha}$	$\frac{(27)}{m_\alpha}$
1													
2													
:													
n													

$\Sigma(20) =$

$$F_2 = \frac{\Sigma(29)}{\Sigma(20) - \Sigma(30)}$$

Repeat steps 24 to 30 incl.

$\Sigma(29) =$

$\Sigma(30) =$

Fig. 35.11. Tabular form for calculating factor of safety of slope by method of slices, if surface of sliding is not circular.

whence

$$\Delta E_n = S \sec \alpha - (W_a + W_b + \Delta T_n) \tan \alpha - zb\gamma_w \tan \alpha \quad (35.32)$$

However, it may also be seen from the force polygon that

$$S = \frac{1}{F} [cl + (P - zl\gamma_w - ul) \tan \phi] = \frac{1}{F} [cl + P' \tan \phi] \quad (35.33)$$

By substituting Eq. 35.27 into 35.33, we obtain

$$S = \frac{1}{F} \cdot \frac{cb + (W_a + W_b + \Delta T_n - ub) \tan \phi}{m_\alpha} = \frac{M}{F} \quad (35.34)$$

and, by using Eq. 35.32 and summing for all the slices,

$$\sum [\Delta E_n + zb\gamma_w \tan \alpha] = \sum \left[\frac{M}{F} \sec \alpha - (W_a + W_b + \Delta T_n) \tan \alpha\right] \quad (35.35)$$

But since

$$\Sigma zb\gamma_w \tan \alpha = \tfrac{1}{2}\gamma_w d^2$$

Eq. 35.31 requires the left-hand side of Eq. 35.35 to be zero. Hence the forces ΔT_n must satisfy not only Eq. 25.30, but also

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$$\sum \left[\frac{M}{F} \sec \alpha - (W_a + W_b + \Delta T_n) \tan \alpha\right] = 0 \quad (35.36)$$

Because the problem is statically indeterminate, any set of values T_n satisfying Eqs. 35.30 and 35.36 will assure compliance with all conditions for equilibrium of the slide as a whole and for the horizontal and vertical equilibrium of each slice. However, not all such sets of values are reasonable or possible. For example, the values of T_n must not exceed the shearing strength of the soil along the vertical boundary of the corresponding slice under the influence of the normal force E_n. Moreover, tensile stresses should not occur across a significant portion of any vertical boundary between slices. In most instances it will prove satisfactory and expedient to assign arbitrary but reasonable values to the earth pressure E_n, and on the basis of these values and Eq. 16.5 to calculate approximate upper limiting values for T_n. By trial and error, smaller values of T_n are established that satisfy Eqs. 35.30 and 35.36. A systematic tabular arrangement (Fig. 35.12) is helpful. Values so obtained are substituted into Eq. 35.29. If F differs appreciably from the value determined previously, a revision by successive approximations is indicated. The revision may require alteration of the quantities T_n because of the dependence of M (Eq. 35.34) on F.

Col.		13	14	9	10	15	16	17	31			32	33	34	35	36
Slice	T_n	ΔT_n	$W_a + W_b + \Delta T_n$	cb	ub	(14)–(10)	(15) tan ϕ	(9) + (16)	(14) tan α	F	m_α	$\frac{(17)}{m_\alpha} = M$	$\frac{M}{F}$	sec α	$\frac{M}{F}$ sec α	(35)–(31)
1																
2																
:																
n																

$\Sigma(13)$ ——— $= 0$ $\Sigma(36)$ ——— $= 0$

Fig. 35.12. Tabular form for determining consistent set of shear forces T_n between slices, for substitution into Eq. 35.29, if values of ΔT_n are not considered to be zero.

There is, of course, no assurance that the value of F finally determined by this procedure is correct, because other consistent sets of T-values lead to other factors of safety. However, the values of F for different but reasonable sets of forces between the slices are not likely to differ to a great extent.

It may also be noted that the force polygon (Fig. 35.9*c*) presupposes that each slice is in equilibrium with respect to moments, whereas this condition will not generally be satisfied by the forces derived from the solution. This requirement can be added to those represented by Eqs. 35.30 and 35.36 but the difficulties of calculation are increased substantially. The use of electronic computation is virtually mandatory (Morgenstern and Price 1965).

If the subsoil contains one or more thin exceptionally weak strata, the surface of sliding is likely to consist of three or more sections that do not merge smoothly one into another. In stability computations such a surface cannot be replaced by a continuous curve without the introduction of an error on the unsafe side.

Figure 35.13 represents a slope underlain by a thin layer of very soft clay with cohesion c. If such a slope fails, the slip occurs along some composite surface *abcd*. In the right-hand part of the sliding mass, represented by the area *abf*, active failure must be expected because the earth stretches horizontally under the influence of its own weight. The central part *bcef* moves to the left under the influence of the active pressure on *bf*. The left-hand part of the sliding mass *cde* experiences passive failure due to the thrust of the advancing central part *bcef*.

The first step in investigating the conditions for the stability of the slope is to compute the passive earth pressure P_P of the soil located on the left side of a tentatively selected vertical section *ec* located near the toe of the slope. It is conservative to assume that P_P acts in the horizontal direction. The next step is to estimate the position of the right-hand boundary b of the horizontal part *cb* of the potential surface of sliding and to compute the active earth pressure P_A on a vertical section *fb* through b. The tendency for the mass *bcef* to move to the left

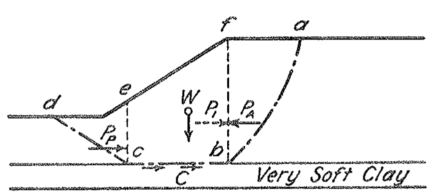

Fig. 35.13. Failure of slope underlain by thin layer of very soft clay.

is resisted by the passive earth pressure P_P and the total cohesion C along bc. If the slope is stable, the sum of these resisting forces must be greater than the active earth pressure P_A which is assumed to act in a horizontal direction. The factor of safety against sliding is equal to the ratio between the sum of the resisting forces and the force P_A. The investigation must be repeated for different positions of the points c and b until the surface of least resistance to sliding is found that corresponds to the least factor of safety.

Problems

1. A wide cut was made in a stratum of soft clay that had a level surface. The sides of the cut rose at 30° to the horizontal. Bedrock was located at a depth of 40 ft below the original ground surface. When the cut reached a depth of 25 ft, failure occurred. If the unit weight of the clay was 120 lb/ft^3, what was its average cohesive strength? What was the character of the surface of sliding? At what distance from the foot of the slope did the surface of sliding intersect the bottom of the excavation?

Ans. 500 lb/ft^2; midpoint circle; 18 ft.

2. The rock surface referred to in problem 1 was located at a depth of 30 ft below the original ground surface. What were the average cohesive strength of the clay and the character of the surface of sliding?

Ans. 450 lb/ft^2; toe circle.

3. A cut is to be excavated in soft clay to a depth of 30 ft. The material has a unit weight of 114 lb/ft^3 and a cohesion of 700 lb/ft^2. A hard layer underlies the soft layer at a depth of 40 ft below the original ground surface. What is the slope angle at which failure is likely to occur? 57

Ans. $\beta = 69°$.

4. A trench with sides rising at 80° to the horizontal is excavated in a soft clay which weighs 120 lb/ft^3 and has a cohesion of 250 lb/ft^2. To what depth can the excavation be carried before the sides cave in? At what distance from the upper edge of the slope will the surface of sliding intersect the ground surface?

Ans. 9 ft; 8 ft.

5. A bed of clay consists of three horizontal strata, each 15 ft thick. The values for c for the upper, middle, and lower strata are, respectively, 600, 400, and 3000 lb/ft^2. The unit weight is 115 lb/ft^3. A cut is excavated with side slopes of 1 (vertical) to 3 (horizontal) to a depth of 20 ft. What is the factor of safety of the slope against failure?

Ans. 1.2.

6. To what depth can the trench in problem 4 be excavated without bracing if the soil has, in addition to its cohesion, an angle of internal friction of 20°?

Ans. 14.2 ft.

Selected Reading

A detailed discussion of the method of slices and the assumptions on which it is based may be found in Taylor, D. W. (1948): *Fundamentals of soil mechanics,* New York, John Wiley and Sons, pp. 432–441. A condensed summary of the method from the point of view of effective stress and the use of pore-pressure coefficients is given in Bishop, A. W. (1955): "The use of the slip circle in the stability analysis of slopes," *Géot.,* **5,** pp. 7–17.

Charts for the solution of many cases of practical importance are contained in Bishop, A. W. and N. R. Morgenstern (1960): "Stability coefficients for earth slopes," *Géot.,* **10,** pp. 129–150. Solutions for many other cases are given by Janbu, N. (1954*b*): "Stability analysis of slopes with dimensionless parameters," *Harvard Soil Mech. Series No. 46,* 81 pp.

The most general analysis available, not restricted to a circular surface of sliding and considering the forces between slices, is developed mathematically by Morgenstern, N. R. and V. E. Price (1965): "The analysis of the stability of general slip surfaces," *Géot.,* **15,** pp. 79–93. An electronic computer is needed for the solution.

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CONTENTS

CHART SOLUTIONS FOR ANALYSIS OF EARTH SLOPES

John H. Hunter, Department of Civil Engineering, Virginia Polytechnic Institute and State University; and
Robert L. Schuster, Department of Civil Engineering, University of Idaho

This paper compiles practical chart solutions for the slope stability problem and is concerned with the use of the solutions rather than with their derivations. Authors introduced are Taylor, Bishop and Morgenstern, Morgenstern, Spencer, Hunter, and Hunter and Schuster. Many of the solutions introduced appeared originally in publications not commonly used by highway engineers. In addition to the working assumptions and parameter definitions of each writer, the working charts are introduced, and example problems are included. The chart solutions cover a wide variety of conditions. They may be used to rapidly investigate preliminary designs and to obtain reasonable estimates of parameters for more detailed packaged computer solutions; in some cases, they may be used in the final design process.

• THE FIRST to make a valid slope stability analysis possible through use of simple charts and simple equations was Taylor (9). With the advent of high-speed electronic computers, other generalized solutions with different basic assumptions have been obtained and published. Unfortunately, these chart solutions have been published in several different sources, some of which are not commonly used by highway engineers in this country. This paper introduces several of these solutions that may prove useful and deals with how to use these solutions rather than with their derivations.

These chart solutions provide the engineer with a rapid means of determining the factor of safety during the early stages of a project when several alternative schemes are being investigated. In some cases they can be used in the final design procedure. Chart solutions such as these may very well serve as preliminary solutions for more detailed packaged computer software programs that are widely available (12).

Those chart solutions that appear to be most applicable to highway engineering problems involving stability of embankment slopes and cut slopes are presented here. In addition to introducing some solutions that may be unfamiliar, this compilation provides a quick means of locating various solutions so that rapid comparisons of advantages and disadvantages of each solution can be made.

The presentation of each solution includes pertinent references and contains sections on calculation techniques, working assumptions and definitions, limitations of the approach, and an example problem. In each case only a sufficient number of curves have been shown to indicate the scope of the charts and to illustrate the solutions. The reader should refer to the appropriate references for greater detail.

TAYLOR SOLUTION

The solution found by Taylor (9, 10) is based on the friction circle (ϕ circle) method of analysis and his resulting charts are based on total stresses. Taylor made the following assumptions for his solution:

1. A plane slope intersects horizontal planes at top and bottom. This is called a simple slope.

Sponsored by Committee on Embankments and Earth Slopes and presented at the 50th Annual Meeting.

2. The charts assume a circular trace for the failure surface.
3. The soil is an unlayered homogeneous, isotropic material.
4. The shear strength follows Coulomb's Law so that $s = c + p \tan \phi$.
5. The cohesion, c, is constant with depth as is shown in Figure 1(a).
6. Pore pressures are accounted for in the total stress assumption; therefore, seepage need not be considered.
7. If the cross section investigated holds for a running length of roughly two or more times the trace of the potential rupture surface, it is probable that this, a two-dimensional analysis, is valid.
8. The stability number in the charts is that used by Terzaghi and Peck (11) in presenting Taylor's solution. The stability number, N, is $\gamma H_c/c$.
9. The depth factor, D, as shown in Figure 2, is the depth to a firm stratum divided by the height of the slope.

The following limitations should be observed in using Taylor's solution:

1. It is not applicable to cohesionless soils.
2. It may not be applied to the partial submergence case.
3. Tension cracks are ignored.
4. According to Taylor, his analysis does not apply to stiff, fissured clays.

The charts presented by Terzaghi and Peck for Taylor's solution consist of the following:

1. A chart for soils with $\phi = 0$ deg with depth factors, D, varying from 1.0 to ∞ and slope angles, β, varying from 0 to 90 deg (Fig. 3),
2. A chart for materials having cohesion and friction with ϕ varying from 0 to 25 deg and β varying from 0 to 90 deg (Fig. 4), and
3. A chart for locating the critical circle of a slope failure (not presented in this paper).

Examples of use of Taylor's solution follow:

1. A cut is to be excavated in soft clay to a depth of 30 ft. The soil has a unit weight of 115 pcf and a cohesion of 550 psf. A hard layer underlies the soft layer at a depth of 40 ft below the original ground surface. What is the slope angle, if any, at which failure is likely to occur?

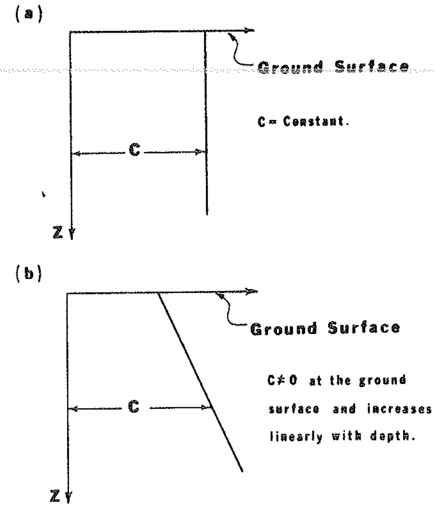

Figure 1. Comparison of assumptions for cohesion, c, as made by various investigators.

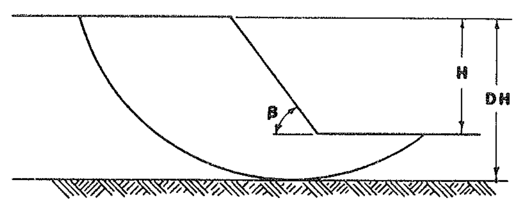

Figure 2. Elements of a simple slope.

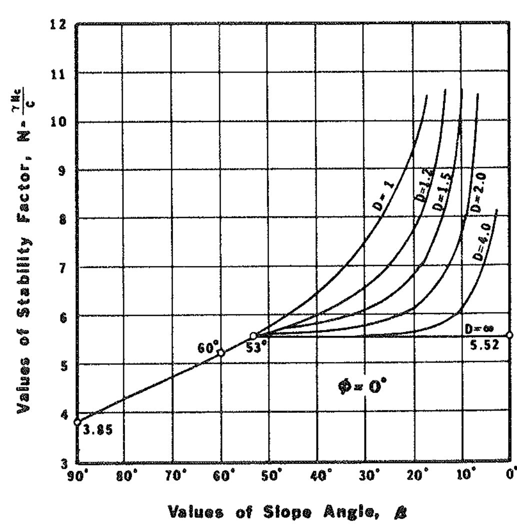

Figure 3. Relations between slope angle, β, and stability number, N, for different values of depth factor, D [after Terzaghi and Peck (11)].

Solution: Because the soil is a soft clay, ϕ is assumed to be zero and the chart of Figure 3 is applicable:

$$D = 40/30 = 1.33$$

If failure is to occur, the critical height, H_c, is 30 ft:

$$N = (\gamma H_c)/c = (115)(30)/550 = 6.28$$

From Figure 3, for D = 1.33 and N = 6.28, β may be read as 30 deg, which is the unknown that was to have been determined.

2. A cut is to be excavated in a material that has a cohesion of 250 psf, a unit weight of 115 pcf, and an angle of shearing resistance of 10 deg. The design calls for a slope angle of 60 deg. What is the maximum depth of cut that can be made and still maintain a factor of safety of 1.5 with respect to the height of the slope?

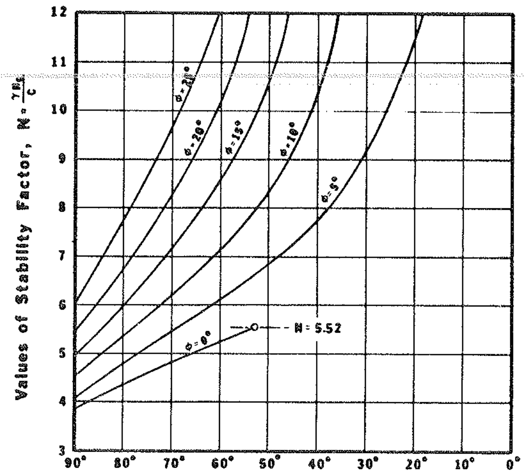

Figure 4. Relations between slope angle, β, and stability number, N, for materials having cohesion and friction, for various values of ϕ [after Terzaghi and Peck (11)].

Solution: Because the soil has both cohesion and angle of shearing resistance, the chart of Figure 4 is applicable. The factor of safety (with respect to height) of 1.5 is the critical height, H_c, divided by the actual height, H. The depth factor, D, does not enter into the solution if the soil is a c, ϕ type of soil.

From Figure 4, for ϕ = 10 deg and β = 60 deg, N may be read as 7.25. From the definition of stability number,

$$N = (\gamma)(H_c)/c$$

or

$$H_c = (c)(N)/\gamma = (250)(7.25)/115 = 15.75 \text{ ft}$$

$$H = H_c/1.5 = 15.75/1.5 = 10.5 \text{ ft}$$

Thus, it would be possible to make a 60-deg cut at any depth up to 10.5 ft and still maintain a factor of safety that is equal to or greater than 1.5.

BISHOP AND MORGENSTERN SOLUTION

Bishop's adaptation of the Swedish slice method (1) was used by Bishop and Morgenstern (2) for their solution. Their charts are based on effective stresses rather than total stresses. Consequently, it is necessary to take pore pressures into consideration.

Bishop and Morgenstern made the following assumptions:

1. The geometry of the slope is simple, as was the case for Taylor's solution. The potential sliding surface is assumed to be cylindrical; the trace of the sliding surface is assumed to be a portion of a circle.
2. The pore pressure is accounted for by use of the pore pressure ratio, r_u. This ratio is defined as being equal to $u/(\gamma h)$, where h = depth of point in soil mass below the soil surface, γ = unit weight of the soil (bulk density), and u = pore pressure of water in the soil. The pore pressure ratio is assumed to be constant throughout the cross section; this is called a homogeneous pore pressure distribution. If there are minor variations in r_u throughout the cross section, an average value of r_u can be used.
3. For steady-state seepage, use a weighted average of r_u over the section.
4. The factor of safety, FS, is defined as m - (n) (r_u), where m and n are determined by using charts in Figures 5 through 7.

5. Depth factors, D, of 1.0, 1.25, and 1.5 are used in this solution where the depth factor is defined as Taylor defined it: The depth to a hard stratum is the depth factor multiplied by the embankment height.

6. The solution implies that the cohesion is constant with depth as shown in Figure 1(a). An interesting feature of this solution is that pore pressures can be changed to see what effect this will have on the stability of the slope.

Bishop and Morgenstern's solution has the following limitations:

1. There is no provision for intermediate water table levels.

2. The averaging technique for pore pressure ratio tends to give an overestimation of the factor of safety. In an extreme case, this overestimation will be on the order of 7 percent.

In using this chart solution it is convenient to select the critical depth factor by use of the lines of equal pore pressure

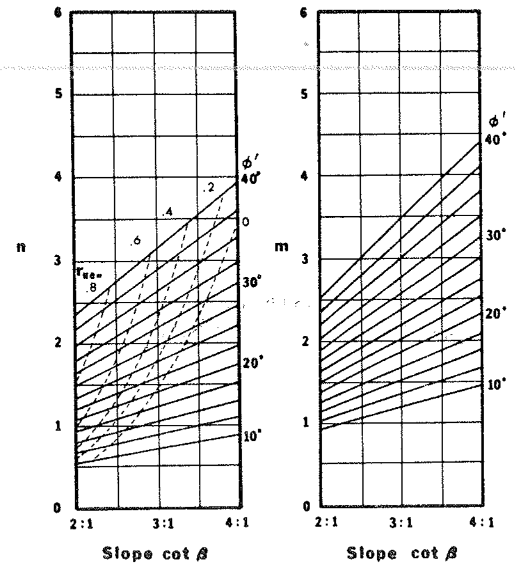

Figure 5. Stability coefficients, m and n, for $\frac{c'}{\gamma H}$ = 0.05 and D = 1.00 [after Bishop and Morgenstern (2)].

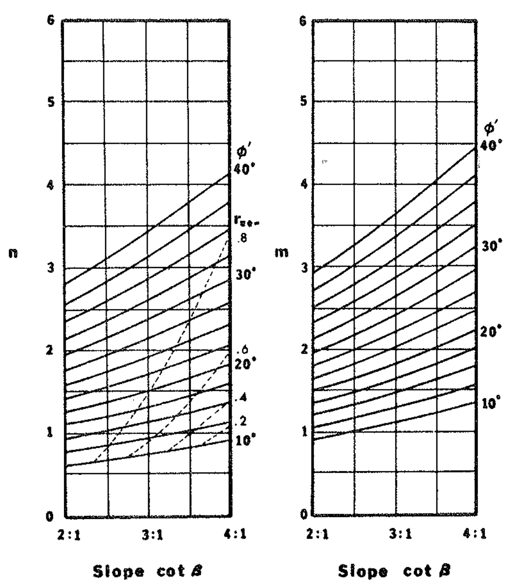

Figure 6. Stability coefficients, m and n, for $\frac{c'}{\gamma H}$ = 0.05 and D = 1.25 [after Bishop and Morgenstern (2)].

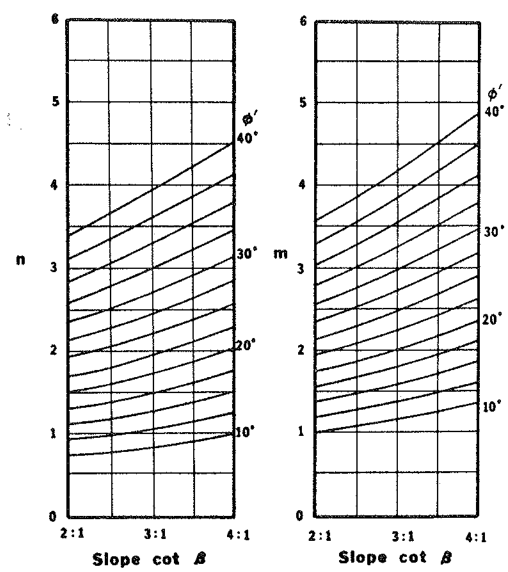

Figure 7. Stability coefficients, m and n, for $\frac{c'}{\gamma H}$ = 0.05 and D = 1.50 [after Bishop and Morgenstern (2)].

ratio, r_{ue}, on the charts of Figures 5 and 6. The ratio, r_{ue}, is defined as $(m_2 - m_1)/(n_2 - n_1)$ where n_2 and m_2 are values for a higher depth factor, D_2, and n_1 and m_1 correspond to a lower value of the depth factor, D_1.

If the design value of pore pressure ratio is higher than r_{ue} for the given section and strength parameters, then the factor of safety determined with the higher depth factor, D_2, has a lower value than the factor of safety determined with the lower depth factor, D_1. This is useful to know when no hard stratum exists or when checking to see if a more critical circle exists not in contact with a hard stratum. The example problem will clarify this concept.

To determine the minimum factor of safety for sections not located directly on a hard stratum, enter the appropriate chart for the given $c'/(\gamma H)$ and, initially, for D = 1.00. Note that c' is the effective stress value of cohesion and H is the height of the slope while γ is the unit weight of the soil. The values of β and ϕ' define a point on the curves of n with which is associated a value of r_{ue} given by the dashed lines. If that value is less than the design value of r_u, the next depth factor, D = 1.25, will yield a more critical value of the factor of safety. If, from the chart for D = 1.25, the values are checked and r_{ue} is still less than the design value for r_u, move to the chart for D = 1.50 with the same value of $c'/(\gamma H)$.

Bishop and Morgenstern (2) show charts for values of $c'/(\gamma H)$ of 0.00, 0.025, and 0.05 with depth factor, D, values of 1.00, 1.25, and 1.50. Only enough charts are shown here to illustrate the solution.

An example of Bishop and Morgenstern's solution follows:

A slope is cut so that the cotangent of the slope angle, β, is 4.0. The cut is 140 ft deep. A hard stratum exists at a depth of 60 ft below the bottom of the cut. The soil has an effective angle of shearing resistance, ϕ', of 30 deg. The effective cohesion, c', is 770 psf. The unit weight is 110 pcf, and it is estimated that the pore pressure ratio, r_u, is 0.50 for the slope. 65

From the given conditions, $c'/(\gamma H) = 770/(110)(140) = 0.050$. From Figure 5, for D = 1.00 with $c'/(\gamma H) = 0.050$, $\phi' = 30$ deg, and cot $\beta = 4.0$, it is seen that $r_{ue} < 0.5$. Therefore, D = 1.25 is the more critical value for depth factor. Using Figure 6, with the same value of $c'/(\gamma H)$ and with D = 1.25, it is found that $r_{ue} > 0.5$. In this case the maximum value that D could have is $(140 + 60)/140 = 1.43$. Therefore, within the limitations of the charts, D = 1.25 is the critical depth factor. From Figure 6 it is seen that m = 3.22 and n = 2.82 for the given values of $c'/(\gamma H)$, ϕ', and cot β. Accordingly, the following factor of safety is obtained:

$$FS = m - (n)(r_u) = 3.22 - 2.82(0.50) = 1.81$$

The chart for D = 1.50 for $c'/(\gamma H) = 0.050$ (Fig. 7) is not necessary for the solution to this example problem, but it is given to indicate the range in this particular sequence of charts.

MORGENSTERN SOLUTION

Morgenstern (6) used Bishop's adaptation of the Swedish slice method of analysis (1) to develop a solution to the slope stability problem that is somewhat different from the one he developed with Bishop. His solution is, again, based on effective stresses rather than total stresses. His solution is primarily for earth dams, but there are highway cuts and fills that nearly fulfill his assumptions. Morgenstern made the following assumptions:

1. The slope is a simple slope of homogeneous material resting on a rigid impermeable layer at the toe of the slope.
2. The soil composing the slope has effective stress parameters c' (cohesion) and ϕ' (angle of shearing resistance), both of which remain constant with depth.
3. The slope is completely flooded prior to drawdown; a full submergence condition exists.

4. The pore pressure ratio $\overline{\beta}$, which is $\Delta u/\Delta \alpha_1$, is assumed to be unity during drawdown, and no dissipation of pore pressure occurs during drawdown.

5. The unit weight of the soil (bulk density), γ, is assumed to be constant at twice the unit weight of water of 124.8 pcf.

6. The pore pressure can be approximated by the product of the height of soil above a given point and the unit weight of water.

7. The drawdown ratio is defined as L/H where L is the amount of drawdown and H is the original height of the slope.

8. To be consistent, all assumed potential sliding circles must be tangent to the base of the section. This means that the value of H in the stability number, $c'/(\gamma H)$, and in L/H must be adjusted for intermediate levels of tangency (see the example problem for clarification).

Morgenstern's solution is particularly good for small dams and consequently might be particularly applicable where a highway embankment is used as an earth dam or for flooding that might occur behind a highway fill. Another important attribute of the method is that it permits partial drawdown conditions.

This method is somewhat limited by its strong orientation toward earth dams. If a core exists, it is noted that this violates the assumption of a homogeneous material. Another limitation is the assumption that the unit weight is fixed at 124.8 pcf. Attention is also called to the assumption of an impermeable base.

Morgenstern's charts cover a range of stability numbers, $c'/(\gamma H)$, from 0.0125 to 0.050 and slopes of 2:1 to 5:1. The maximum value of ϕ' shown on his charts is 40 deg. Following are some example problems using Morgenstern's method:

1. An embankment has a height, H, of 100 ft. It is composed of a soil with an effective cohesion, c', of 312 psf and an effective angle of shearing resistance, ϕ', of 30 deg. The unit weight of the soil must be assumed to be equal to 124.8 pcf. The embankment is to have a slope so that the cotangent of the slope angle is 3.0. What is the minimum factor of safety for the complete drawdown condition?

66

Solution: The stability number, $c'/(\gamma H) = 312/[(124.8)(100)] = 0.025$. With this value and with $\cot \beta = 3.0$, $\phi' = 30$ deg, and the drawdown ratio $L/H = 1.0$, the factor of safety is directly obtainable from Figure 9 as FS = 1.20. By examining the charts in Figures 8 through 10, it can be seen that the critical circle is tangent to the base of the slope;

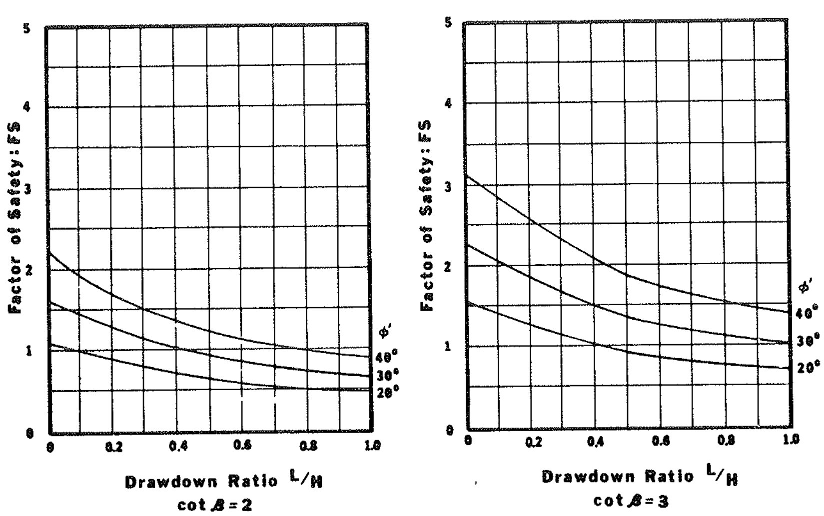

Figure 8. Relationship between factor of safety, FS, and drawdown ratio, L/H, for $\frac{c'}{\gamma H} = 0.0125$ [after Morgenstern (6)].

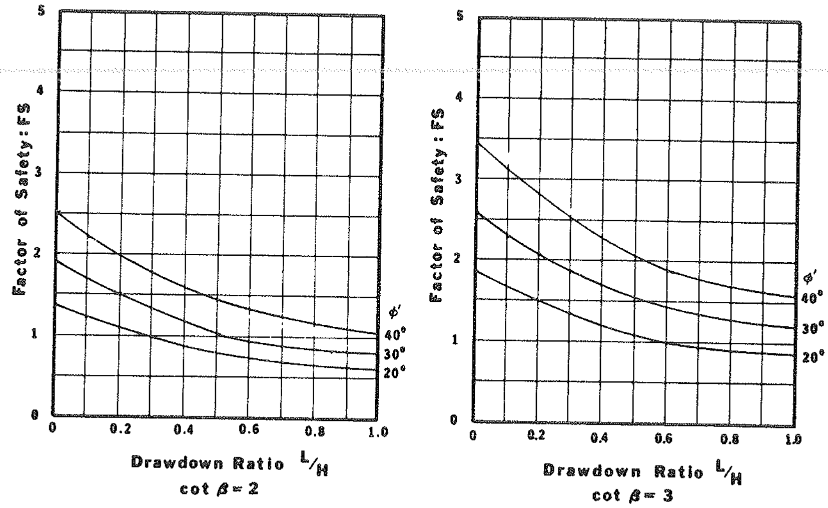

Figure 9. Relationship between factor of safety, FS, and drawdown ratio, L/H, for $\frac{c'}{\gamma H} = 0.025$ [after Morgenstern (6)].

if any other tangency is assumed, H would have to be reduced. If H is reduced, then the stability number is increased and this will, in all cases, result in a higher factor of safety.

2. It is now required to find the minimum factor of safety for a drawdown to mid-height of the section in the prior example.

Solution a: Considering slip circles tangential to the base of the slope, the effective height of the section, H_e, is equal to its actual height and the stability number remains

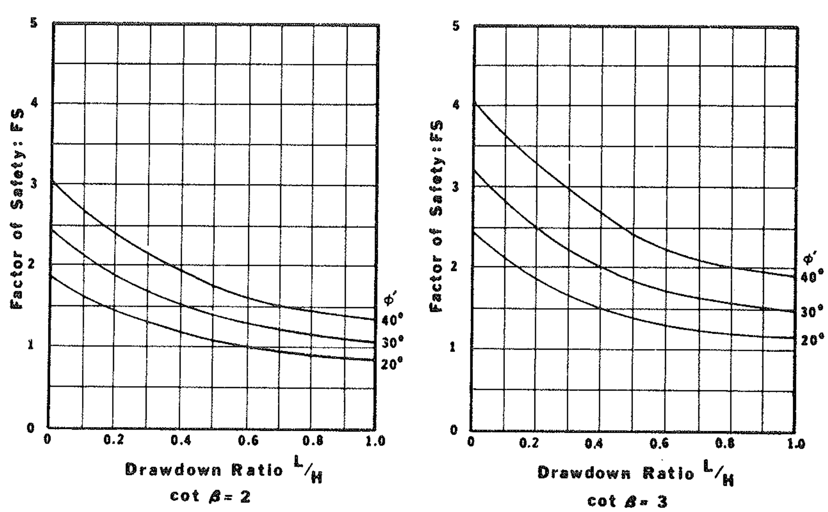

Figure 10. Relationship between factor of safety, FS, and drawdown ration, L/H, for $\frac{c'}{\gamma H} = 0.050$ [after Morgenstern (6)].

unchanged as 0.024. With this value of stability number and L/H_e = 0.50, and with other conditions remaining the same, the factor of safety may be read from Figure 9 as FS = 1.52.

Solution b: Considering slip circles tangential to mid-height of the slope, the effective height is equal to one-half the actual height so that $H_e = H/2 = 100/2 = 50$ ft. Thus $c'/(\gamma H_e)$ is twice that of the previous solution or 0.05, and $L/H_e = 1.00$. The minimum factor of safety, as determined by Morgenstern's solution, can be read directly from Figure 10 as FS = 1.48.

Solution c: Considering slip circles tangential to a level H/4 above the base of the slope, H_e becomes $3H/4 = 75$ ft. Thus the stability number $c'/(\gamma H_e) = 0.033$, and L/H_e = 0.67. The minimum factor of safety for this family must be obtained by interpolation. From Figure 9 with $c'/(\gamma H_e) = 0.025$, the factor of safety is 1.31, and from Figure 10 with $c'/(\gamma H_e) = 0.05$, the factor of safety is 1.61. Interpolating linearly for $c'/(\gamma H_e)$ = 0.033, the minimum factor of safety for this family is $1.31 + 0.30/3 = 1.41$.

These examples demonstrate that for partial drawdown the critical circle may often lie above the base of the slope, and it is important to investigate several levels of tangency for the maximum drawdown level. In the case of complete drawdown, the minimum factor of safety is always associated with circles tangent to the base of the slope and the factor of safety at intermediate levels of drawdown need not be investigated. This may not be the case if the pore pressure distribution during drawdown differs significantly from that assumed by Morgenstern.

SPENCER SOLUTION

Bishop's adaptation of the Swedish slice method has been used by Spencer (8) to find a generalized solution to the slope stability problem. Spencer assumed parallel interslice forces. His solution is based on effective stresses. Spencer defines the factor of safety, FS, as the quotient of shear strength available divided by the shear strength mobilized.

Spencer made the following additional assumptions and definitions for his solution:

1. The soils in the cut or embankment and underneath the slope are uniform and have similar properties.
2. The slope is simple and the potential slip surface is circular in profile.
3. A hard or firm stratum is at a great depth, or the depth factor, D, is very large.
4. The effects of tension cracks, if any, are ignored.
5. A homogeneous pore pressure distribution is assumed with the pore pressure coefficient, r_u, equal to $u/(\gamma h)$, where u = mean pore water pressure on base of slice, γ = unit weight of the soil (bulk density), and h = mean height of a slice.
6. The stability number N is defined as $c'/[(FS)\gamma H]$.
7. The mobilized angle of shearing resistance, ϕ'_m, is the angle whose tangent is $(\tan \phi')/FS$.

Spencer's method does not prohibit the slip surface from extending below the toe. His solution permits the safe slope for an embankment of a given height to be found rapidly.

Although the limitations of Spencer's method are few, it is noted that a simple trial and error solution is required to find the factor of safety with the slope and soil properties known. In addition, it is difficult to use his method for intermediate levels of the water table. Spencer provides charts for a range of stability number, N, from 0.00 to 0.12 with mobilized angle of shearing resistance varying from 10 to 40 deg and slope angles up to 34 deg. Charts are provided for pore pressure ratio, r_u, with values of 0.0, 0.025, and 0.50. Only one of these charts (Fig. 11) is shown for use in the example problem. Spencer furnishes charts for locating the critical surface.

An example of Spencer's solution follows:

An embankment is to be formed with a factor of safety of 1.5 and a height of 100 ft. The soil has an effective cohesion of 870 psf and an effective angle of shearing resistance of 26 deg. The unit weight of the soil is 120 pcf and the pore pressure ratio is 0.50. Find the slope that corresponds to this factor of safety.

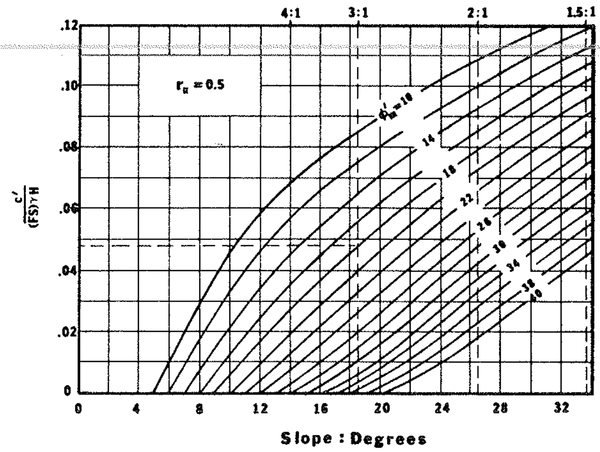

Figure 11. Relationship between stability number $\left[\frac{c'}{(FS)\ \gamma H}\right]$ and slope angle, β, for various values of ϕ'_m [after Spencer (8)].

Solution: The stability number,

$$N = c'/[(FS)\gamma H] = 870/[(1.5)(120)(100)] = 0.048$$ 69

$$\tan \phi'_m = (\tan \phi')/FS = \tan 26 \text{ deg}/1.5 = 0.488/1.5$$

$$\tan \phi'_m = 0.325 \text{ or } \phi'_m = 18 \text{ deg}$$

Referring to Figure 11 for $r_u = 0.50$, the slope corresponding to a stability number of 0.48 and $\phi'_m = 18$ deg is $\beta = 18.4$ deg. This corresponds approximately to a slope of 3:1.

Linear interpolation between charts for slopes for r_u values falling between the chart values is probably sufficiently accurate.

HUNTER SOLUTION

In 1968, Hunter (3) approached the slope stability problem with two assumptions that are different from the solutions previously presented in this paper. He assumed that the trace of the potential slip surface is a logarithmic spiral and the cohesion varies with depth. His charts are based on total stresses. Hunter's working assumptions and definitions follow:

1. The section of a cut is simple with constant slope, and top and bottom surfaces are horizontal.
2. The soil is saturated to the surface through capillarity.
3. The soil is normally consolidated, unfissured clay.
4. The problem is two-dimensional.
5. The shear strength can be described as $s = c + p \tan \phi$ where c varies linearly with depth, as is shown in Figure 1(b). It is assumed that the ratio c/p' is a constant, where p' is the effective vertical stress. Note that p' increases with depth.
6. If $\phi > 0$ deg, the potential slip surface is a logarithmic spiral. If $\phi = 0$ deg, the potential failure surface is a circle because the logarithmic spiral degenerates into a circle for this case.

7. The effective stresses immediately after excavation are the same as those before excavation. This describes the end-of-construction case.
8. The water table ratio, M, is defined as $(h/H)(\gamma_w/\gamma')$, where h = depth from top of slope to the water table during consolidation, H = height of cut, γ_W = unit weight of water, and γ' = submerged or buoyant unit weight of soil.
9. While c increases linearly with depth, the angle of shearing resistance, ϕ, is constant with depth.
10. A stability number, N, is obtained so that the factor of safety,

$$FS = \frac{c}{\gamma z_1 N}$$

where

$$z_1 = z + h\left(\frac{\gamma_w}{\gamma'}\right)$$

and z = depth below the original ground surface of cut to point where cohesion, c, is determined.

An equivalent and perhaps more convenient relationship is

$$FS = \left(\frac{c}{p'}\right)\left(\frac{\gamma'}{\gamma}\right)N$$

70 because often (c/p') can be estimated from Skempton's (7) formula,

$$\left(\frac{c}{p'}\right) = 0.11 + 0.0037(PI)$$

where PI = plasticity index of the soil in percent.

11. The depth ratio, D, is defined the same as in the description of Taylor's work. If $\phi > 0$ deg, the effects of a firm layer at any depth are negligible. If $\phi = 0$ deg, the depth factor can have a significant but small influence on the factor of safety, as is shown in Hunter's (3) work and also by Hunter and Schuster (4). Only when the stability number, N, is greater than about 25 and the slope angle, β, is less than about 15 deg is the small reduction in N important enough to be taken into account.

Hunter's solution permits realistic variation in the values of cohesion, c, for normally consolidated soils. It can easily handle the situation for the water table at any of a wide range of elevations. This solution should be used only for normally consolidated materials.

Numerous charts are furnished by Hunter. The charts show the slope angle, β, varying from 5 to 90 deg, and the angle of shearing resistance ϕ varying from 0 to 35 deg in steps of 5 deg. The water table ratio, M, is varied from 0.00 to 2.00 in steps of 0.25. In addition, many tables and graphs are shown that are useful in locating the critical failure surface. In this paper only one chart (Fig. 12) is shown to illustrate Hunter's solution. An example of Hunter's solution follows:

A 25-ft slope of 30 deg is to be cut in normally consolidated material with a unit weight of 112 pcf and the water table at a depth of 10 ft. The material has been tested (on a total stress basis) and found to have a ϕ of 10 deg with a plasticity index of 25 percent. It is required to estimate the factor of safety of this slope.

Solution: Using Skempton's relationship,

$$c/p' = 0.11 + 0.0037(PI) = 0.11 + 0.0037(25) = 0.2025$$

$$M = \left(\frac{h}{H}\right)\left(\frac{\gamma_w}{\gamma'}\right) = \left(\frac{10}{25}\right)\left(\frac{62.4}{112 - 62.4}\right) = \left(\frac{10}{25}\right)(1.26) = 0.502$$

Using M = 0.50, β = 30 deg, and ϕ = 10 deg, find the stability number from the chart in Figure 12. Read N = 17.1. Thus, the factor of safety,

$$FS = \left(\frac{c}{p'}\right)\left(\frac{\gamma'}{\gamma}\right) N$$

$$FS = (0.2025)\left(\frac{49.6}{112}\right)(17.1) = 1.52$$

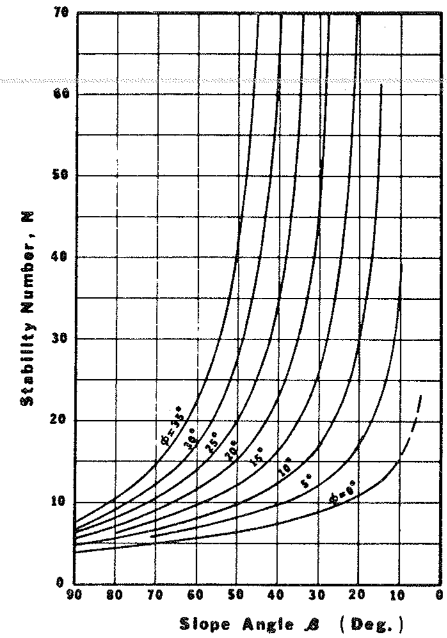

Figure 12. Relationship between slope angle and stability number, N, for M = 0.50 and unlimited depth of soil, for various values of ϕ. Solid lines indicate shallow surfaces and dashed lines indicate deep surfaces [from Hunter (3)].

HUNTER AND SCHUSTER SOLUTION

Based on some of Hunter's original work (3), Hunter and Schuster (4) published a solution for the special case of ϕ = 0 deg in normally consolidated clays. This solution is a total stress solution.

The assumptions are the same as those made by Hunter in the previous section, except that the potential sliding surface is a circular arc rather than a logarithmic spiral. In particular, this solution permits the cohesion, c, to increase linearly with depth, and the saturated soil may have a water table that can be anywhere within a wide range. The depth factor, D, is taken into account. The method ignores tension cracks.

The charts furnished by Hunter and Schuster show the water table ratio varying from 0.00 to 2.00 in steps of 0.25, and the depth ratio, D, varying from 0 to 4. Only those charts (Figs. 13 and 14) necessary to illustrate the example problem are shown. Some example problems using Hunter and Schuster's solution follow:

1. A cut 15 ft deep is to be made in a normally consolidated clay with a slope angle of 30 deg. The water table is 5 ft below the original ground surface. The soil weighs 104 pcf, and the c/p′ ratio is 0.24 for the soil. What is the factor of safety for this cut?

Solution: The water table ratio M is

$$M = \left(\frac{h}{H}\right)\left(\frac{\gamma_w}{\gamma'}\right) = \left(\frac{5}{15}\right)\left(\frac{62.4}{41.6}\right) = 0.50$$

In Figure 13, with M = 0.50 and β = 30 deg, N = 8.9 (a possible shallow failure). Calculate the factor of safety, FS, as

$$FS = \left(\frac{c}{p'}\right)\left(\frac{\gamma'}{\gamma}\right) N = (0.24)\left(\frac{41.6}{104}\right)(8.9) = 0.855 < 1.00$$

It can therefore be concluded that this cut is impossible without failure occurring.

2. A cut at a slope angle of 10 deg is to be made 15 ft deep in a normally consolidated clay with the water table 15 ft from the surface. Underneath the clay at a depth of 30 ft is a harder, stronger stratum. When tested, the soil showed ϕ = 0 deg on a total stress basis. The ratio c/p′ for this soil is 0.24, and its unit weight is 104 pcf. Find the factor of safety for this proposed cut.

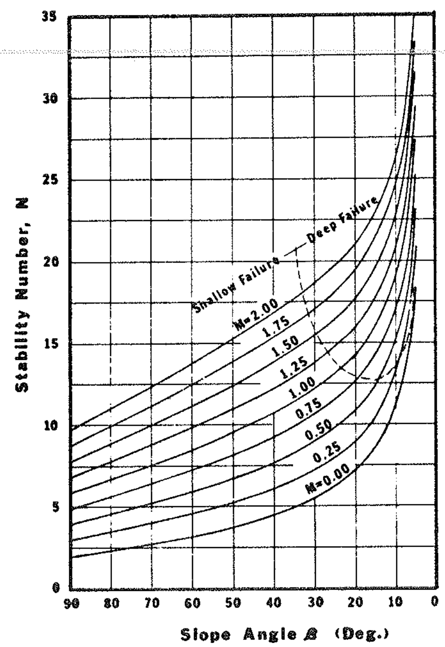

Figure 13. Relationship between slope angle, β, and stability number, N, for $\phi = 0$ and unlimited depth of clay [from Hunter and Schuster (4)].

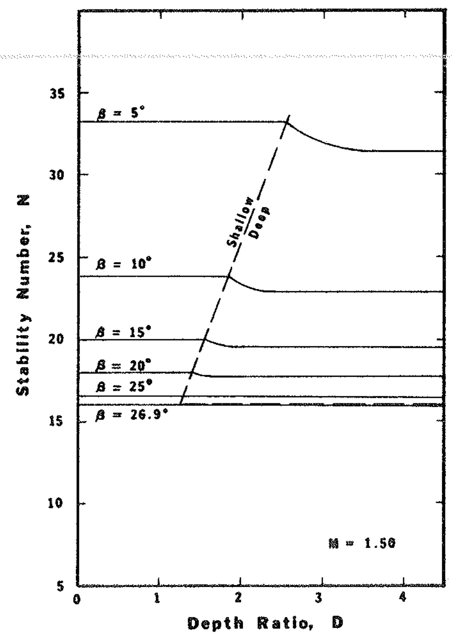

Figure 14. Relationship between depth factor, D, and stability number, N, for $\phi = 0$ for selected values of slope inclination [from Hunter and Schuster (4)].

Solution:

$$M = \left(\frac{h}{H}\right)\left(\frac{\gamma_w}{\gamma'}\right) = \left(\frac{15}{15}\right)\left(\frac{62.4}{41.6}\right) = 1.50$$

From Figure 13, with $\beta = 10$ deg and $M = 1.50$, the value of N plots up in the deep failure zone with a value of approximately 23.9. Because it is in the deep failure zone, $D = 30/15 = 2.0$ may be important.

From Figure 14, for $M = 1.50$, $D = 2.0$, and $\beta = 10$ deg, it is seen that N reduces slightly to 23.2. Thus, the factor of safety is

$$FS = \left(\frac{c}{p'}\right)\left(\frac{\gamma'}{\gamma}\right) N = (0.24)\left(\frac{41.6}{104}\right)(23.2) = 2.23$$

Note that the depth factor, in general, has only a negligible or quite small effect on the factor of safety.

One set of generalized solutions that should be mentioned is that developed by Janbu (5). His solutions are extensive and do not lend themselves to simple presentations as has been the case with the other solutions. Janbu's solutions are useful in analyzing the influence of drawdown conditions and the effect of water-filled tension cracks and surcharge. Janbu implies that both the cohesion, c, and the angle of shearing resistance, ϕ, are constant with depth. Although not reviewed here, Janbu's solutions are recommended to the engineer who frequently deals with stability analyses of slopes.

SUMMARY

The chart solutions developed by Taylor, Bishop and Morgenstern, Morgenstern, Spencer, Hunter, and Hunter and Schuster can be applied to a number of types of slope stability analyses. Some of the methods presented were originally developed only for cuts; some were developed especially for embankments or fills such as earth dams. Each solution presented, however, is applicable to some highway engineering situation. References have been given indicating more complex chart solutions not illustrated here, and an entry into the literature on computerized solutions has been given. Of the solutions introduced, those of Taylor, Hunter, and Hunter and Schuster are best suited to the short-term (end-of-construction) cases where pore pressures are not known and total stress parameters apply. The other methods are intended for use in long-term stability (steady seepage) cases with known effective stress parameters.

The methods make similar assumptions regarding slope geometry, two-dimensional failure, and the angle of shearing resistance being constant with depth. However, they vary considerably in assumptions regarding variation of cohesion, c, with depth, position of the water table, base conditions, drawdown conditions, and shape of the failure surface. Altogether, a wide range of conditions can be approximated by these available generalized solutions.

Each author has attempted to reduce the calculation time required to solve stability problems. The chart solutions alone may be sufficient for many highway problems; in other cases, chart solutions may save expensive computer time by providing a reasonable estimate as a starting point for computer programs that solve slope stability problems.

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REGION 6 SUPPLEMENT NO. 19

Page Code	Place at End of Chapter	No. of Sheets Superseded	New
Index		1	1
50--1 thru --24	50	-	12

Digest:

Incorporates slope design guide. This is a guide which provides general values or recommendations for cut and fill slope ratios. Data needed to use the guide are soil classifications, general field conditions in respect to density and moisture, and height of cut or fill.

The recommendations given must be modified to fit local conditions and experiences.

T. A. SCHLAPFER
Regional Forester

TRANSPORTATION ENGINEERING HANDBOOK

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*-50 - MATERIALS ENGINEERING

1. Slope Design Guide. This guide provides general values for maximum excavation and embankment slope ratios based on a combination of general field description and the Unified Soil Classification of the material. Standard penetration test values (AASHO T 206) and in-place density (relative compaction) are used to further define the materials.

- -

Explanation of common field conditions and the appropriate use of the following sections, together with sample problems are described in the following sections:

Section 1 - Homogeneous soils

Section 2 - Stratified deposits

Section 3 - Residual soils

Section 4 - Cemented and Special soils

Section 5 - Embankments (Table 1)

Section 6 - Benching

Design charts and tables are given for the following materials:

Section 7 - Coarse grained soils (less than 50 percent passing the #200 sieve).

a. Sands and gravels with nonplastic fines (PI 3 or less)

Table II

Unified Soil Classification:
GW, GP, SW, SP, GM and SM.

b. Sands and gravels with plastic fines (PI greater than 3)

Charts I and II

Unified Soil Classification:
GM, SM, GC, SC and dual class of these and above.-*

*- Section 8 - Fine grained soils (greater than 50 percent passing the #200 sieve)

Charts III and IV

Unified Soil Classification:
ML, MH, CL and CH.

Section 9 - Unweathered rock (Table III)

1. Guidelines for Use and Limitations

a. General. The following preliminary steps are necessary in order to most effectively use this guide. First, all published and file sources of soils, geologic, hydrologic and climatic information pertaining to the area should be reviewed. Certain of these reports are frequently quite specific in identifying, describing and characterizing the various kinds and properties of materials in that area. Maps are often available indicating general or specific location of troublesome or trouble-free areas. This information will greatly help in the next step of identifying and describing, the various soil, geologic and bedrock conditions in detail in the field. Caution must be exercised to characterize the entire cross section of cut or fill area--surface samples are generally not representative. In addition, the depth to water table and locations of seeps and springs and possibility of ponding water against or above slopes should be noted since water is one of the major factors relating to stability. Such study should also recognize seasonal changes in ground water and runoff patterns. This study in most cases would benefit from multi-discipline review including engineering, geologic, soil and hydrologic backgrounds.

Any guide such as this should not be followed indiscriminately as a precise answer to all situations that will be encountered in the field. It is offered as a guide to be used in connection with engineering judgment and analysis. Too many variables and unusual conditions exist that cannot be properly accounted for by this guide. This guide must be used in connection with local experience to arrive at reasonable values for slope ratios. Additional information and discussions of unusual situations can be found in many of the publications listed in the References.

b. Special Limitations. The higher the cut or fill the more critical the need becomes for accurate investigation. The following limitations for cuts or fills apply to the charts and tables of this guide:-*

*-0 to 50 feet in vertical height only a minimal investigation is necessary in noncritical areas. This would include soil classification, some excavation by hand or backhoe, seismic information, and observation of nearby slopes of similar material to identify various soil layers and existing stability conditions. This range in height is indicated by the solid lines on the charts.

50 to 100 feet in vertical height a more intensive investigation is necessary. This would include all of the requirements listed in the 0 to 50 feet situation and may also require test borings in the form of auger or drill holes to definitely identify various layers and the location of water. In most cases an experienced technician or engineer would be required for interpretation of the results. This range in height is indicated by dashed lines on the charts.

Over 100 feet in vertical height the slope should be designed by specialists in soil mechanics using more refined methods than are indicated in this design guide. If specialist assistance is not available at the Forest level, the Regional Office should be consulted. In no case should this design guide be used for slopes over 100 feet in vertical height.

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Special investigation is essential where serious loss of property, extensive resource damage, or loss of life might result from the slope failure or when crossing known areas of slope instability such as existing slides. Some especially troublesome soils may also require special investigation, these would include organic material and soils, swelling clays, layers of weathered schists or shales, talus deposits, pockets of loose, water-bearing sands and silts, fissured clays, and layered geologic deposits where subsurface conditions are impossible to determine from visual or seismic information.

c. Application. Once the soil types have been described, the water sources located, and the limitations of the guide observed, then refer to the appropriate explanation section of the guide (Sections 1 to 6). These sections will indicate how to use the various charts and tables in Sections 7 to 9. Once the proper chart or table is indicated then the maximum height-slope relationship can be determined. In most cases the depth-*

of cut or fill will be known, thus the chart or table will give the maximum slope ratio (horizontal to vertical) that can be-used under the given conditions. This may require a trial and error solution in the case of sloping original ground. If the maximum slope ratio is fixed then the highest cut or fill that can be constructed will be determined from the charts. Obviously the slope ratio will not apply if the recommended slope is equal to or flatter than the slope of the natural ground. This situation would require special investigation as to possible modifications of the ratio or alternate schemes such as structures or relocation.

Revegetation problems are also a necessary consideration in the selection of slope ratios. Normally it is very difficult or impossible to revegetate slopes steeper than 1:1. Generally the steeper the cutbank, the more intensive will be the measures needed to adequately revegetate the slope and protect it from surface erosion. Leaving cut slopes rough will improve the opportunities for seed, mulch and fertilizer to catch. This can improve the chances for erosion control while reducing planting costs.

d. Factor of Safety. The factor of safety is generally expressed as a dimensionless number, with a value greater than one being safe and a value less than one indicating failure. Typical design factors of safety against slope failure are between 1.1 and 2.0, with the lower values used for inexpensive and less permanent construction. The following values are used in the design guide: 81

(1) Table II, sands and gravels with non-plastic fines, is based on the factor of safety with respect to sliding (translation) of approximately 1.1. This factor is proportional to the tangent of the slope angle for a given soil density.

(2) Charts I, II, III and IV, plastic soils, are based on a factor of safety with respect to rotation of 1.5. This assumes the natural ground at the top of the cut to be horizontal. Since this is normally not the case, the factor will be less than 1.5 depending upon the steepness of the natural slope but still above 1.1. The factor of safety is dependent upon the cohesion, height of cut, soil density and slope angle. In Charts I and II it is also dependent upon the angle of

*- internal friction and in Charts III and IV upon the depth to a dense layer.

- -

1. Section 1--Homogeneous Soils. Homogeneous soils are those that do not exhibit layering or stratification of various materials. They have the same properties (gradation, strength, etc.) throughout, though there may be a slight increase in density with depth due to the weight of the overlying material.

If the soil fits this category, then the description can be matched with the appropriate chart or table in Sections 7 to 9, and the answer used directly, tempered only with engineering judgment and local experience.

Problem soils would be loose, saturated sands and soft clays. Loose, saturated sand will liquefy and flow. Shock, vibration from construction equipment or a rapid change in the water table will cause a liquefaction failure. Densifying the sand prior to construction will often solve this problem.

Soft clays will often fail in very shallow cuts, thus their maximum height is limited as indicated by soils #4 and #5 in Charts III and IV. Clays underlain by seams of fine water-bearing sand will often fail by lateral spreading, even though the slope in the clay has been stable for long periods of time. Special investigation is required in this case.

Example Problem 1.

Field conditions. A proposed cut through an old stream deposit consisting of a well graded sandy gravel (non-plastic). The in-place natural density is 115 pcf which is approximately 95 percent of maximum density as determined by AASHO T 99. The cut is to be 40 feet deep with ground water expected to be one fourth the way up the cut during the wettest period.

Recommended slope. The material is best described by Section 7A, soil #1 of Table II. The density is somewhat in between the loose and dense state. By interpolation this would give a slope of 1:1 for low ground water and 2-1/3:1 for high ground water conditions. Since the ground water will be only a quarter of the way up the final slope (actually the final seepage path will vary), a value of 1.3:1 would normally be recommended (graphical solution, table II), however a value of 1.4:1 should be used to account for the lower unit weight of the granular soil.-*

*-Example Problem 2.

Field conditions. A proposed cut in a transported clayey sand (SC) with a plasticity index of 15 and a liquid limit of 20. The soil is of intermediate density (90 percent relative compaction weighing approximately 125 pcf) and can be dug fairly easy with a shovel. The cut is to be 40 feet deep with the water table estimated at 20 feet below the surface. No evidence of stratification is present.

Recommended slope. The soil is best described in Section 7B, soil #3. Chart I gives a maximum slope of 1.5:1 and Chart II a value of 3.2:1. By interpolation a value of 2.3:1 is recommended as the water table is approximately half way up the proposed slope.

2. Section 2--Stratified Deposits. These are deposits that consist of layers of various materials usually transported by water or wind action and deposited in horizontal or dipping layers. They may consist of alternate layers of clay and sand or silt and underlain by bedrock, and are typical in former lake basins, stream valleys, glacial regions, etc.

The sand or silt layers in many cases are sources of water, resulting in seepage on the face of the final cut. These layers are likely to wash out and are affected by frost action resulting in sloughing. They may also bring about hydrostatic pressure from water that is trapped in them during wet weather, resulting in failure of the entire slope. Horizontal drains by means of perforated pipes are a possible solution to this problem. Spacing and the location of these should be based on more extensive investigation and analysis by specialists.

Soft or fissured clay layers will also cause stability problems due to their low strength. Special investigation is also warranted in this situation.

The design height of stable slopes in stratified deposits using the design guide are based on the height from the top of the cut to the bottom of the exposed layer in question. Thus in the case of a sand layer over a clay layer, the clay slope will be based not on the thickness of clay exposed, but on the total height of the cut. Refer to example problem #3 in Section 3 on residual soils for specific calculations of a similar situation.-*

*- Stratified deposits that dip more than 20° to 30° toward the cut will present additional stability problems as they may fail along the boundaries between layers. These slopes will require special stabilization techniques such as drainage, or retaining walls of rock, wood or metal.

3. Section 3--Residual Soils. Residual soils have been formed by rock weathering in place thus retaining much of the original structural and bedding plane orientation of the parent rock. They are usually composed of three fairly well defined layers:

Layer 1: Residual soil (A and B horizon material).

Layer 2: Weathered rock (C horizon material).

Layer 3: Unweathered rock (D horizon material).

Cuts in residual material are often stable with slopes steeper than transported soils and thus can be as steep as 45° to 80° (there are exceptions). The main requirement is that the dip of bedding planes towards the proposed cut be less than the residual angle of shearing resistance. This is approximately 20° to 30° for weathered rock and 30° to 45° for unweathered roack. For rock dipping at steeper angles the shearing resistance along the contacts becomes the controlling factor.

For most cases the following guidelines are suggested:

Layer 1: Use the descriptions that apply to Charts I and II for coarse grained soils in Section 7B, or Charts III and IV for fine grained soils in Section 8.

Layer 2: Use soil #1 on Charts I and II if the material is dense with no complete joint system. Use soils #2 or #3 for all other cases. (The descriptions in Section 7, item b. may not apply).

Layer 3: Use the appropriate rock type in Table III Section 9.

Be sure to account for highwater table or poor internal drainage in the coarse grained soils by using Chart II. Section 7B. In any case the height of cut for any layer is measured from the top of the excavation and not just the thickness of the individual layer.-*

*-Some material such as decomposed granite will ravel continuously due to weathering, filling ditch lines. Flattening the slope usually does not control this problem unless vegetation can be established. A successful solution is to step the slope in small benches, say two feet wide and with height determined by the slope. The material will then ravel, filling the benches and at the same time protecting the underlying material, aiding plant growth to start and preventing further raveling. This procedure can be applied to many types of material (references 9 and 10).

In badly jointed or weathered rock some means of removing the hazard must be provided. This can be done by:

- Rock bolting.

- Grouting of fissures.

- Horizontal berms or a wide ditch to catch falling rock.

- Fences, wire mesh, or walls to catch falling rock.

- Pneumatically applied motar to prevent raveling.

In addition, the prevention of water pressure buildup in seams must be prevented by the proper drainage or sealing of the entrances. Any design must start with a complete picture of the joints, fissures, and bedding planes of the area of concern.

Example Problem 3.

Field conditions: A residual soil on the west side of the Cascades has an A and B horizon of 25 feet of plastic clay of firm consistency (less than 10 percent rock particles are evident in this layer); a C horizon of weathered shale 20 feet thick and then unweathered shale bedrock. The bedding planes in the weathered and unweathered shale dip approximately 5° toward the proposed cut. No ground water is evident; however, the clay soil is moist. The shale is somewhat fractured. The proposed cut is to be 55 feet deep.

Recommended slope. First the appropriate soil charts must be determined. Sections 8, Chart III, soil #3 described the clay A and B horizon soil; Section 7B, Chart I, soil #2 applies to the weathered shale (even though the description in Section 7, item b. does not apply) and the unweathered rock values in Section 9, Table III, applies to the shale bedrock.-*

* From Chart III, soil #3 a slope of 1:1 is recommended for the 25 foot clay portion. The total depth to the bottom of the weathered shale portion of the cut is 45 feet, thus a slope of 0.85:1 is recommended from Chart I, soil #2. Table III recommends a slope between 1/2:1 and 3/4:1 for shale bedrock. Since the bedrock is fractured and the dip angle is flat, a slope of 3/4:1 is chosen.

A compound slope of 3/4:1 - 0.85:1 - 1:1 may be constructed, however since the three slopes are relatively close to the same value, a total slope ratio of 1:1 may be selected for construction ease. In the case of compound slopes, benches at each soil change are sometimes recommended. Extreme care is essential if a lower layer is to be constructed at a slope ratio flatter than an overlying layer. Such designs should be reviewed by a materials engineer due to the danger of overloading the lower layer.

4. Section 4--Cemented and Special Soils. Soils such as cemented loess (wind blown silt), glowing avalanche pumice (nuee ardente), volcanic tuff, caliche, etc., will often stand near vertical (1/4:1). Sloughing results when water (rain or surface runoff) dissolves or softens the cementing agent or weathering dislodges individual particles. Near vertical cuts will minimize this problem. Local experience is the best indicator.

Do not place partially saturated sands or silts in this category (noncemented) as they will slough and eventually stabilize with slopes indicated in the section on granular material with nonplastic fines (Table I). This condition can be identified by drying a sample of the soil and observing if the structure collapses when slightly disturbed.

5. Section 5--Embankments. Embankment strength can be defined with a reasonable degree of certainty when they are placed with moisture and density control. Those with no compaction control (an undesirable procedure) should be placed to eliminate potential failure planes, seepage channels, large voids and other stability hazards.

The following table is based on Section 7 and 8. Slopes subject to periods of inundation are those that might pond water, act as reservoir slopes or back up water around a culvert entrance. The table indicates maximum allowable slopes.-*

*-TABLE I

Unified Class or Description	Slope not subject to inundation	Slope subject to inundation	Minimum percent compaction (AASHO T 99)
Hard angular blasted or ripped rock	1.2:1	1.5:1	
GW	1.3:1	1.8:1	90 1/
GP, SW	1.5:1	2:1	90 1/
GM, GC, SP	1.8:1	3:1	90 1/
SM, SC 2/	Chart I, soil 3	Chart II, soil 3	90
	Chart I, soil 4	Chart II, soil 4	no control
ML, CL 2/	Chart I, soil 4	Chart II, soil 4	90
	Chart I, soil 5	Chart II, soil 5	no control
MH, CH 2/	Chart III, soil 3	Chart III, soil 4	90
	Chart III, soil 4	Chart III, soil 5	no control

In certain cases the displacement or settlement of the foundation material upon which the embankment is to be built will be the critical design consideration. In these situations the slope of the embankment may be based on the allowable bearing capacity of the foundation material. Soft or organic layers should be analyzed to prevent these failures. A materials engineer should be consulted in these cases.

1/ With no compaction control flatten slopes approximately 25 percent.

2/ Do not use any slope steeper than 1 1/2:1 for these soil types.-*

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*-Example Problem 4.

Field conditions. A fill is to be constructed from a silty soil with liquid limit of 40 (ML). The final fill may impound water due to a critical culvert placed under the middle of the fill. At this point the fill will be 30 feet high. Ninety percent relative compaction (AASHO T 99) is proposed for construction.

Recommended slope. Section 7B, Chart II, Soil 4 is the appropriate chart since the soil will be subject to inundation due to the culvert entrance conditions. A slope of 3.8:1 is recommended for this critical condition at 90 percent relative compaction, if water will be impounded to the top of the fill.

Under normal conditions (no impoundment of water) the recommended fill slope would be 1.8:1 (Chart I, Soil 4). If the water was only expected to be impounded halfway up the slope, then an interpolated value of 2.8:1 could be used.

6. Section 6--Benching. Benching of cut slopes is done to reduce erosion, catch ravel, provide a berm to install drainage, etc. They generally have no advantage over flattening the slope in providing slope stability. The quantity of earthwork and the factor of safety for stability are approximately the same if the location of the top of the cut is the same.

a. Noncohesive soils. Since stability of slope is not height dependent, it is not possible to use benches without increasing earthwork volume since the slope of each section cannot be steeper than the maximum allowable for the composite slope. It is possible to reduce erosion because velocity of flow is reduced by shortening the flow path, however ravel can generally be handled cheaper by using a widened ditch section. Since these soils are free-draining, midslope drainage for stability is seldom required unless layering or stratification exists causing concentration of water in a layer.

b. Cohesive soils. The stability of slopes in these soils are height dependent. The height of each bench must be designed individually for stability. The composite slope must also be checked. This can be done by using the appropriate chart to determine the required slope for each bench section individually and then checking the overall composite slope measured along a line from ditch to catch point.-*

*- Benching in cohesive soils may be done for any of the reasons discussed in the previous sections. Since earthwork quantities do not increase as compared to flattening the slope, it is easier to justify than with noncohesive soils. In erosion control it should be kept in mind that steepening the slope makes revegetation more difficult.

c. Rock. Benches in rock slopes should be designed by a specialist. If not properly designed benches will not catch rock fall but cause them to bounce into the roadway, creating a hazard for traffic.

d. Maintenance. Benches, particularly insloped benches, require maintenance. Designed access for equipment is mandatory for insloped benches. If yearly maintenance is not feasible, benches should always be outsloped. Outsloping is important where impounded water may cause stability problems in fine grained or plastic material.

e. Design. Vertical spacing between benches depends upon the material, with 20 to 50 feet a minimum spacing. Minimum width should be 8 to 10 feet to allow room for equipment to operate effectively for maintenance. A more detailed discussion appears in references #3, pages 122-124, 162-169 and #11, pages 11-36 to 11-40.

7. Section 7--Coarse Grained Soils.

a. Sands and Gravels with Nonplastic Fines. These are soils with less than 50 percent passing the #200 sieve and nonplastic (PI 3 or less). Slopes in these materials are determined by the angularity and interlock between grains, and thus are independent of height. Low ground water conditions will probably be most common, however springs, heavy rainfall, etc., will cause the high ground water condition.-*

NOTE: Use Table II in Graphical Form for solution of slopes in Section 7.a. materials.

*-TABLE II

Soil #	Description	Maximum Slope Ratio (h:v)			
		Low ground water (below bottom of excavation)		High ground water 1/ (Seepage from entire slope)	
		loose 2/	dense 3/	loose 2/	dense 3/
1.	Sandy gravels (GW, GP)	$1\frac{1}{2}$:1	.85:1	3:1	1 3/4:1
2.	Sand, angular grains, well graded (SW)	1.6:1	1:1	3.2:1	2:1
3.	Silty gravels (GM); uniform sands (SP); and silty sands (SM)	2:1	$1\frac{1}{2}$:1	4:1	3:1

1/ Based on material of saturated density approximately 125 pcf. Flatter slopes should be used for lower density material and steeper slopes can be used for higher density material. For every 5% change in density change the ratio by approximately 5%.

2/ Approximately 85 percent of maximum density relative to AASHO T 99.

3/ Approximately 100 percent of maximum density relative to AASHO T 99. -*

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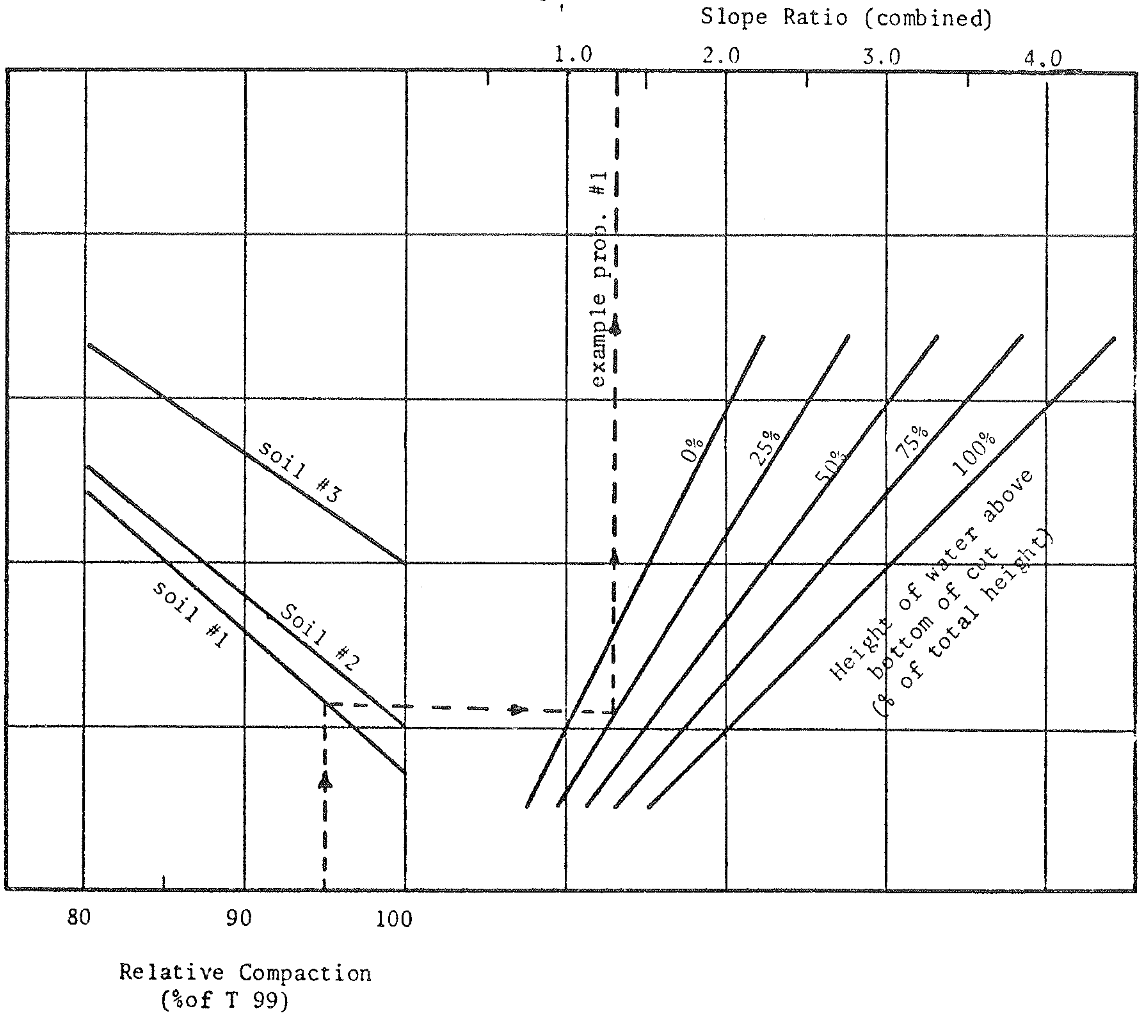
(Cont'd on next printed page)

TRANSPORTATION ENGINEERING HANDBOOK

*- Homogeneous Soils

TABLE II IN GRAPHICAL FORM

COARSE GRAINED SOILS
(nonplastic)



*- Material with intermediate density and/or ground water condition can use interpolated values for the slope ratio.

The standard penetration test blow count per foot is a good indicator for the above slope ratios. Coarse grained soils (+3/4") will give high reading, especially if large gravel or boulders are encountered. Be sure to allow for these variations. The following ranges from loose to dense for each soil type can be expected (generally independent of moisture content):

Soil #	Range of blows (No. per foot) Loose Dense
1.	25 to 60 (refusal) blows per foot
2.	20 to 50 blows per foot
3.	5 to 25 blows per foot

b. Sands and Gravels with Plastic Fines. These are soils with plastic binder (PI greater than 3), thus making the factor of safety height dependent (i.e. the higher the cut, the flatter the maximum stable slope). Five general soil types have been selected. Based on these descriptions, Charts I and II are then used to determine the maximum height-slope relationship. Soils that do not directly fit these five types should be based on an interpolated value.

Chart I is based on low water table (below the bottom of the final excavation) even though the soil may be moist from capillary rise.

Chart II is based on high water table or a completely saturated condition which may occur during periods of heavy rainfall or where water is ponded behind the slope (typical of rapid construction where no drainage has taken place).

Soils with intermediate water table or with steady seepage such as from springs and seeps should use values interpolated between the two tables.

Soil classification will be GM, GC, SM, SC and dual classes including these such as GW-GC, SP-SC, etc.-*

*-Soil number and description: 1/, 2/ Use with Charts I and II.

1. Well-graded material with agular granular particles. Extremely dense and compact (in excess of 100 percent of AASHO T 99 relative compaction) with fines that cannot be molded by hand when moist. Difficult if not impossible to dig with a shovel. May need to be ripped during construction. Standard penetration test blow count greater than 40 blows per foot.

2. Poorly graded material with rounded or low percentage of granular particles. Dense and compact (approximately 100 percent of AASHO T 99 relative compaction) with fines being difficult to mold by hand when moist. Difficult to dig with a shovel. Standard penetration test approximately 30 blows per foot.

3. Fairly well graded material with subangular granular particles. Intermediate density and compactness (approximately 90 to 95 percent of AASHO T 99 relative compaction) with fines that can easily be molded by hand when moist (PI greater than 10). Easy to dig with a shovel. Standard penetration test blow count approximately 20 blows per foot.

4. Well graded material with angular granular particles. Loose to intermediate density (approximately 85 to 90 percent of AASHO T 99 relative compaction). Low plasticity fines (PI less than 10). Extremely easy to dig. Standard penetration test blow count less than 10 blows per foot.

5. Poorly graded material with rounded or low percentage (50-60 percent) of granular particles. Loose density (less than 85 percent of AASHO T 99 relative compaction). Low plasticity fines (PI less than 10). Extremely easy to dig even with the hands. Standard penetration test blow count below 5 blows per foot.

1/ Based on material with a moist density of 125 pcf. Flatter slopes should be used with heavier soils. For every 5 percent increase in density reduce the slope ratio or height approximately 10 percent.

2/ Large gravel and boulders will give misleading results as to the ease of digging and standard penetration test results.-*

*-CHART I

Coarse Grained Soils with Plastic Fines
(Low Water Conditions)

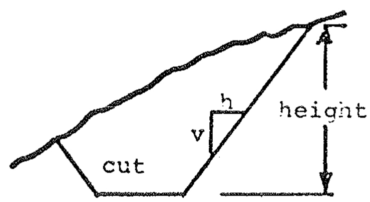
Each curve indicates the maximum height or the steepest slope that can be used for the given soil type.

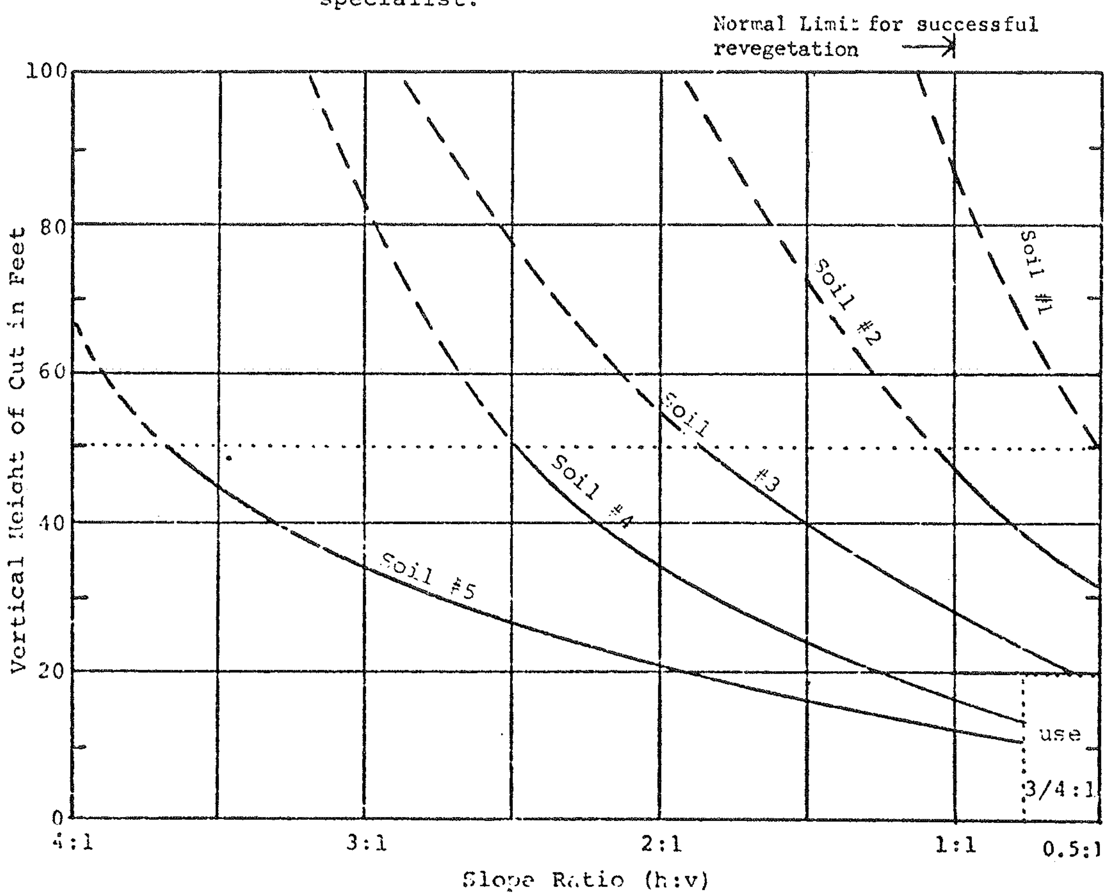
Height limitations:

0-50 feet: minimal investigation.

50-100 feet: intensive investigation.

over 100 feet: investigation by specialist.





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*-February 1973
R-6 Supplement No. 19-*

*- CHART II

Coarse Grained Soils with Plastic Fines
(High Water Conditions)

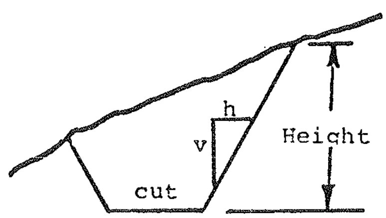
Each curve indicates the maximum cut height or the steepest slope that can be used for the given soil type.

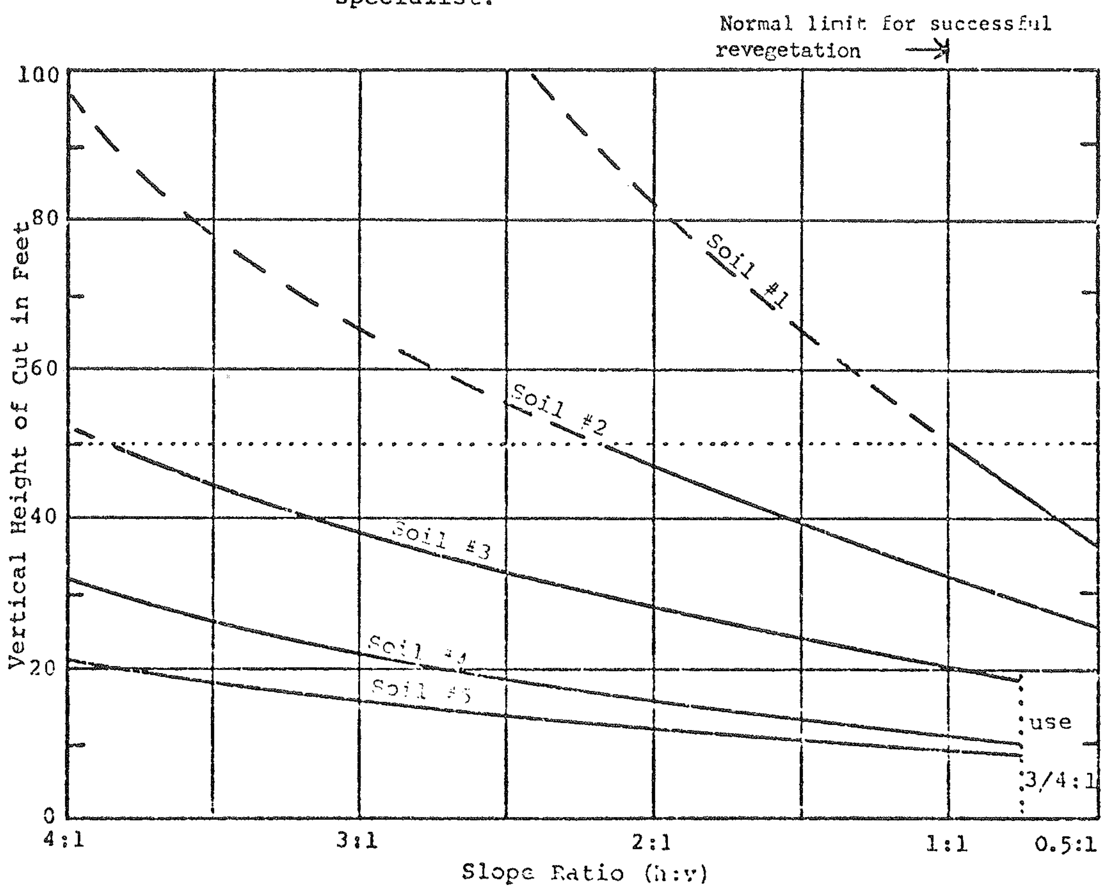
Height limitations:

0-50 feet: minimal investigation.

50-100 feet: intensive investigation.

over 100 feet: investigation by specialist.





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*- 8. Section 8--Fine Grained Soils. These are soils with greater than 50 percent passing the #200 sieve. Soil classification will be ML, MH, CL and CH. 1/

Five soil types have been selected based on consistency or the ability to be molded when moist (field conditions). Complete saturation with no drainage during construction is assumed. Under these conditions the main variable to be considered is the depth to a hard underlying layer such as bedrock or an unweathered residual material. Charts III and IV are used to determine the maximum height-slope relationship. Solutions for stability analysis for soils that do not directly fit these types or bedrock depth should be based on an interpolated values.

In general, the deeper a hard layer exists in a cut, the flatter the maximum allowable slope in the overlying material. A hard layer is identified as one having at least one soil number lower on the chart than the overlying material. A rock ledge is always considered a hard layer.

Chart III assumes a hard layer at the bottom of the proposed excavation. 2/ This chart can also be used for a hard layer part way up the slope. See section 2 for more information.

Chart IV assumes a hard layer at great depth below the bottom of the proposed excavation (at a depth greater than three times the depth of excavation as measured from the bottom of the excavation). For intermediate depths to a hard layer, interpolate between the two charts.

Soil number and description: 3/

1. Very stiff consistency. The soil can be dented only slightly by finger pressure. Ripping may be necessary during construction. Standard penetration test blow count greater than 25 blows per foot.

2. Stiff consistency. The soil can be dented by strong pressure of fingers. Might be removed by spading. Standard penetration test blow count approximately 20 blows per foot.

1/ For ML soils with PI 3 or less, use slopes as defined by soil #3 in Table II.

2/ Based on material with a moist density of 125 pcf; flatter slopes should be used with heavier soils. For every 5 percent change in density change the slope ratio or height approximately 5 percent.

3/ Moisture should not be adjusted when checking consistency. An undisturbed sample shall be used taken at a depth great enough to represent constant moisture content throughout the various seasons.-*

*- 3. Firm consistency. The soil can be molded by strong pressure of fingers. Standard penetration test blow count approximately 10 blows per foot.

4. Soft consistency. The soil can easily be molded by fingers. Standard penetration test blow count approximately 5 blows per foot.

5. Very soft consistency. The soil squeezes between fingers when fist is closed. Standard penetration test blow count less than 2 blows per foot.-*

CHART III

Fine Grained Soils
(Dense Layer at Bottom of Cut)

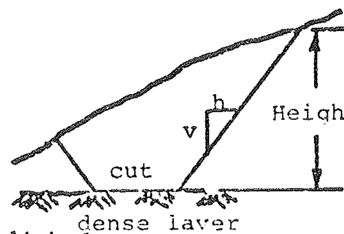
Each curve indicates the maximum vertical cut height or the steepest slope that can be used for the given soil type.

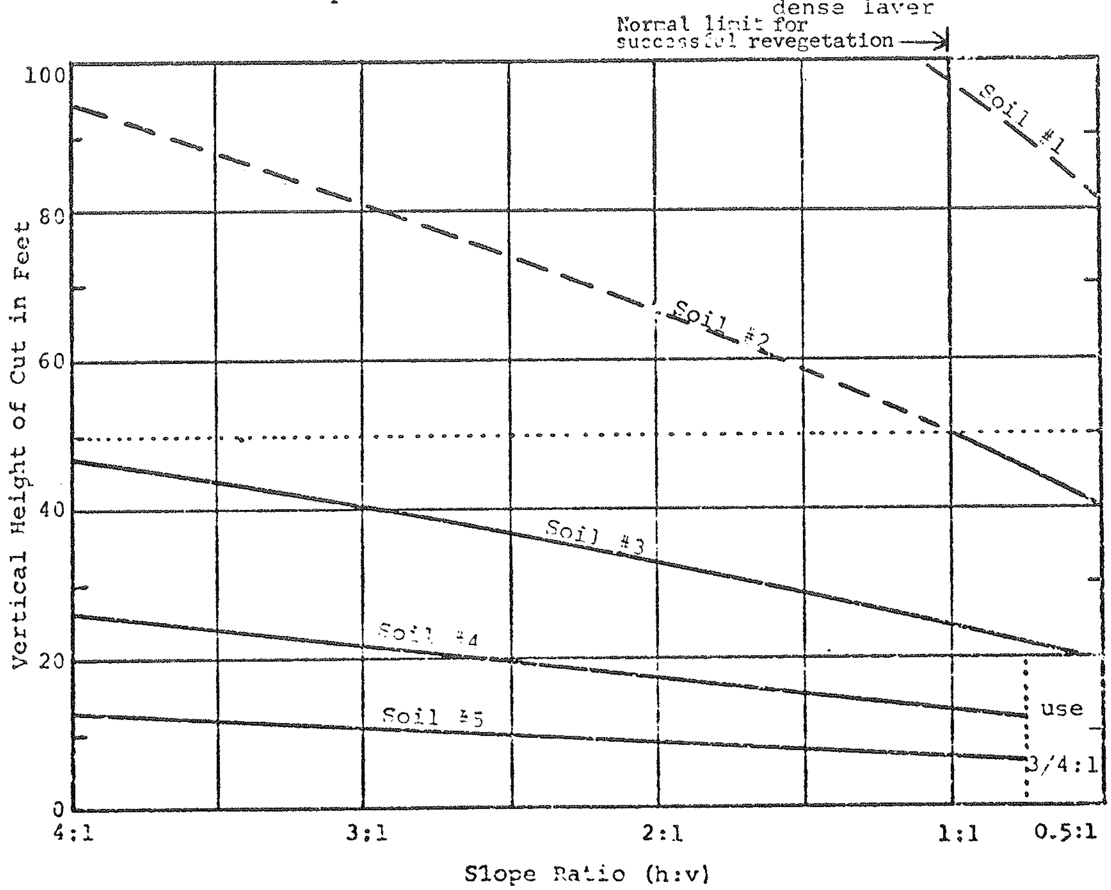
Height limitations:

0-50 feet: minimal investigation.

50-100 feet: intensive investigation.

over 100 feet: investigation by specialist.





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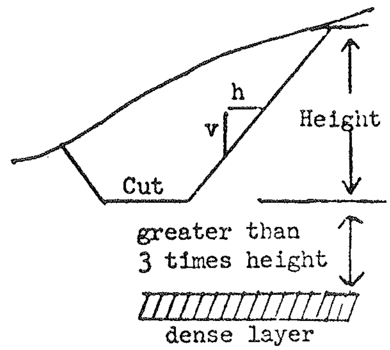
*-CHART IV

Fine Grained Soils
(Dense Layer at Great Depth)



Soil Type	Maximum Height 1/	Slope Ratio (h:v)
1	80	0.5:1
2 2/	40	0.5:1
3 2/	20	0.5:1
4 3/	10	1:1
5 3/	5	1:1

1/ If it is necessary to exceed this height consult with a materials engineer. Benching will not improve stability as these slopes are practically independent of the slope ratio.

2/ If the slope of the natural ground exceeds 20°, then the natural slope may be unstable. A detailed field investigation is necessary to check this condition prior to any design or construction.

3/ If the slope of the natural ground exceeds 10°, then the natural slope may be unstable. A detailed field investigation is necessary to check this condition prior to any design or construction.-*

*-9. Section 9--Unweathered Rock. The stability of rock slopes is dependent upon the material type, the dip of the bedding planes, joints, fractures or faults towards the proposed cut, the type and condition of material in these openings, and the weathering of newly exposed material. If these planes of weakness dip greater than approximately 30° towards the excavation, then the shearing resistance along these planes becomes the critical factor. The exception is when the dip is steeper than the cut slope. If this is the case use the table values. The excavation method can also cause stability problems, especially excessive blasting.

Average values for bedrock excavation with planes of weakness dipping less than 30° are given below. Use the guidelines listed under residual material for weathered rock (Section 3). Consult with a materials engineer for a special investigation when the dip exceeds 30° towards the excavation or if the contact between layers contains plastic or clayer material.

TABLE III

Average Slope Values for Bedrock Excavation

Description	Maximum Slope Range	
	Massive	Fractured
1. Igneous		
Granite, trap, basalt, and volcanic tuff	1/4:1 to 1/2:1	
	See Section 7	
2. Sedimentary		
Massive sandstone and limestone	1/4:1 to 1/2:1	
Interbedded sandstone, shale and lime-stone	1/2:1 to 3/4:1	
Massive clay stone silt stone	3/4:1 to 1:1	
3. Metamorphic		
Gneiss, schist and marble	1/4:1 to 1/2:1	
Slate	1/2:1 to 3/4:1	
Serpentine	Special investigation-*	

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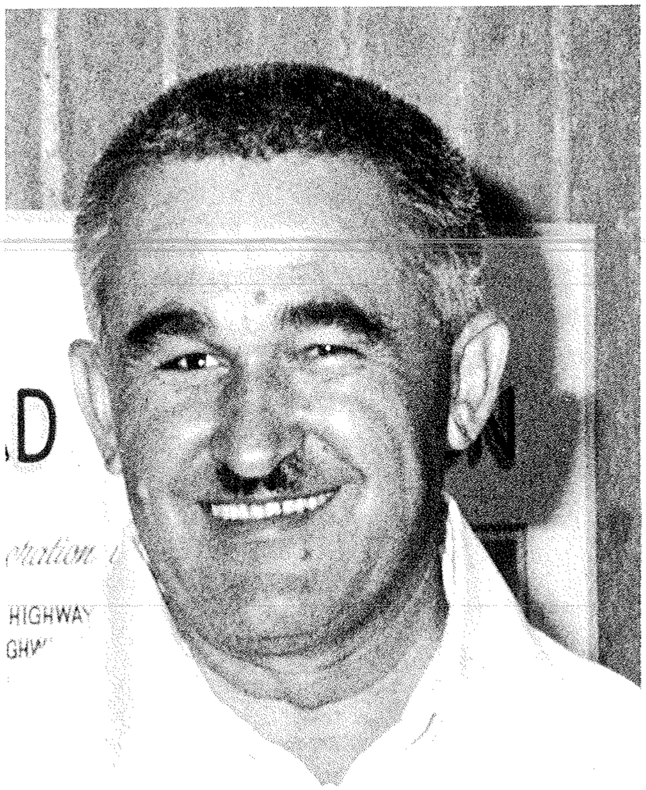

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Analysis of Landslides

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Washington, D. C.

Contents

Determining Corrective Action for Highway Landslide Problems

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State Road Commission of West Virginia

THE PROBLEM of landslides has plagued highway departments throughout the country for many years. For some states, and particularly West Virginia, the damage caused by earth movements represents a major expenditure, one that involves hundreds of thousands of dollars annually. Over 80 percent of the area of West Virginia is located in a landslide-susceptible area. The total number of landslides on the state highways has never been established. However, the writer estimates that this total will approach 1,000 on the 31,000 miles of primary and secondary roads in the state.

The complexities of the landslide problem have very few parallels in highway engineering. The literature on the subject carries numerous references to case histories, but none outlines a systematic, complete approach to the solution of a given problem. The recent bibliography published by the Highway Research Board (1) offers a complete summary of the publications relative to mass movements. The work of geologists on landslides has been and is of considerable value. The classification systems suggested by Sharpe (2) and Ladd (3) assist tremendously in understanding the complicated variety of movements that occur. From the viewpoint of corrective actions, the report by Ladd is perhaps the most comprehensive contained in the landslide literature. Numerous engineers (4, 5, 6, 7, 8, 9, 10, 11, 15) have discussed the application of the theories of Soil Mechanics to the analysis of the stability of a landslide, but there are few details concerning the determination of the effect of a corrective action in terms of stability.

The study that led to the following theory was designed to prepare an approach to the analysis and correction of highway problems dealing with landslides in unconsolidated materials. The primary emphasis was to be towards the correction of existing problems. However, it was felt that the principles should be applicable to the problem of design.

The basis for the study was the writer's experiences in West Virginia, combined with general theories from geology, soil mechanics, and highway engineering. The analysis as advanced is for consideration in the study of all landslides in unconsolidated material, with the exception of those of the nature of fluvial transported material, i. e., the water present is far in excess of normal soil moisture, and the debris is a "relatively small proportion of the flowing mass" (2).

Since one of the primary aims of the study was to consider the applicability of the various corrective measures, the investigation could have been accomplished by a study of existing landslides that have been treated. Such an approach was used by Price and Lilly (12) in 1942. However, a direct study was impossible since as a routine department function there were requests to investigate over 100 landslides during the past three years. Due to a personnel shortage, the demand necessitated superficial analyses but it was decided that the program lent itself to the development of a procedure to evaluate the movements. In addition, it became possible to study the applicability and usefulness of various corrective methods. The theories advanced in the following are not complete for three vital factors remain in the evaluation: (1) observation of those landslides that have been corrected by the methods outlined herein; (2) a more comprehensive study of flow movements,

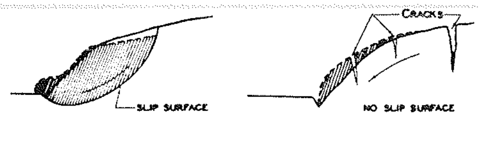

Figure 1. Differentiation between slide and flow (after Sharpe). A slide is a movement of a block of material, whereas flow is entirely internal deformation.

and (3) investigation of less costly methods for correcting landslide problems.

The writer is aware that the analysis is an over-simplification. Extensive study and evaluation is still very necessary, but for the immediate future a working tool is available.

DEFINITIONS

The following definitions will be used throughout. Some of the terms may be argumentative and general, but it is the opinion of the writer that the following are most applicable to the engineering phases of the landslide problem.

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Landslides have been defined by Terzaghi (3) as follows: "The term landslides refers to a rapid displacement of a mass of rock, residual soil, or settlement adjoining a slope in which the center of gravity of the moving mass advances in a downward and outward direction." It will be noted that the time element is involved in the definition only by the term "rapid displacement."

The terms slip-plane, slip-surface, and surface of failure will be synonymous and will refer to the surface that separates the mass in motion from the underlying stable material.

Permanent solutions will be defined as corrections with an anticipated life of at least 50 years. An expedient solution will be considered adequate for a period of a few months to 5 to 10 years.

All corrective actions will be classed as one of two types, elimination or control. The actions involving elimination depend generally upon avoiding or removing the landslide. Control methods are defined as corrections which produce a static condition of the landslide for a finite period of time.

While there have been many classification systems proposed, the bases for the classifications have most generally been related to cause and effect of the movement rather than the mechanics. One notable exception is the system proposed by Hennes (7). For a quantitative analysis of a design or correction for a given landslide, the most satisfactory classifi-

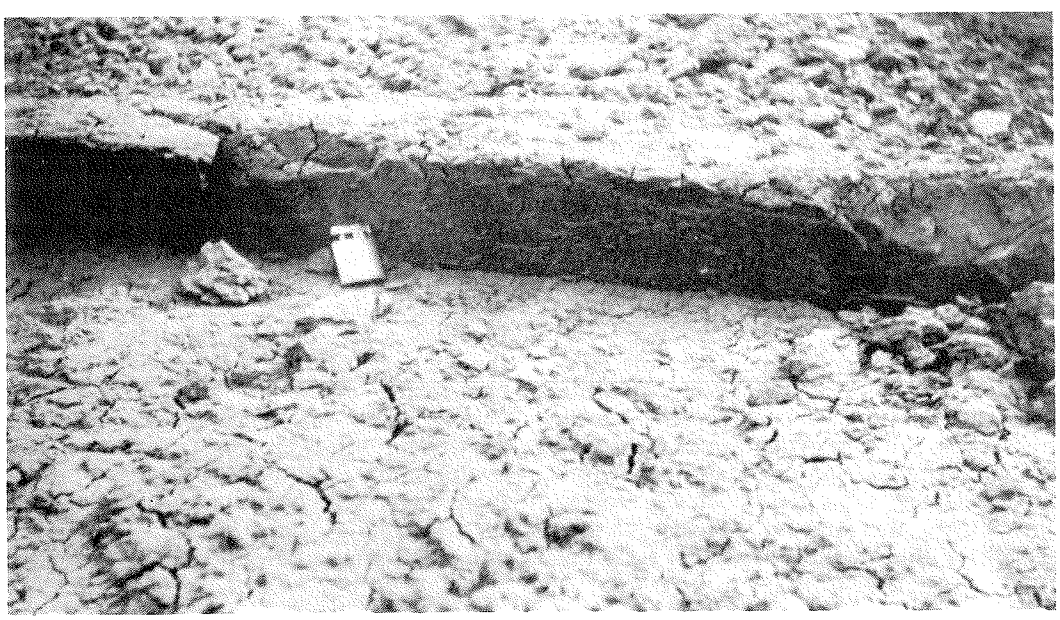


Figure 2. A definite slip-plane, identifying the movement as a slide.

cation is one which differentiates on a basis of the effect of the forces and resistances at work. Thus, the major primary classification would appear to be landslides in consolidated materials, and those in unconsolidated materials. A second primary differentiation would divide the movements into those with a slip-surface and those without a slip-surface. This latter grouping was outlined by Sharpe (2), who termed the former as slides and the latter as flows. The principle is pictured in Figure 1. The movements in flow conditions are the result of internal deformations.

stressed beyond their "fundamental strengths," and as a result, slow but constant internal deformations occur.

BASIC FUNDAMENTALS IN LANDSLIDE ANALYSES

From the observation of landslides in West Virginia, and from a review of the literature on landslides and soil mechanics, the following statements have been outlined by the writer (13) as being fundamental to the analysis of a landslide relative to its correction as a highway problem. It should be empha-

Figure 3. Typical flow movement. Note the characteristic roll of the material at the toe. Some movements originate as a flow and develop into a slide.

For the purpose of the following analyses, the term slide (Fig. 2) will be defined as all landslides which involve unconsolidated material in which the movement is along a slip-surface. The terms flow and creep will be defined as those movements which do not have a slip-surface, the movement resulting from internal deformation. A flow (Fig. 3) will be further defined as being caused primarily by excessive water. The term creep will be differentiated in accordance with Terzaghi's concept (3) that failure occurs at a considerable depth due to the load of the overlying material. The layers at the deeper elevations are

sized that the statements apply primarily to highway problems and may not be of value from an academic viewpoint or for landslide analyses for other purposes.

1. There are numerous instances where the control of the landslide will not be the best solution. For instances that involve the use of an elimination corrective action that avoids the landslide, halting the movement is not generally a factor in the solution (Fig. 4).

2. Determination of "the" cause of a landslide is not always essential to an accurate solution to a highway landslide problem, and is always secondary in importance to an understanding of the

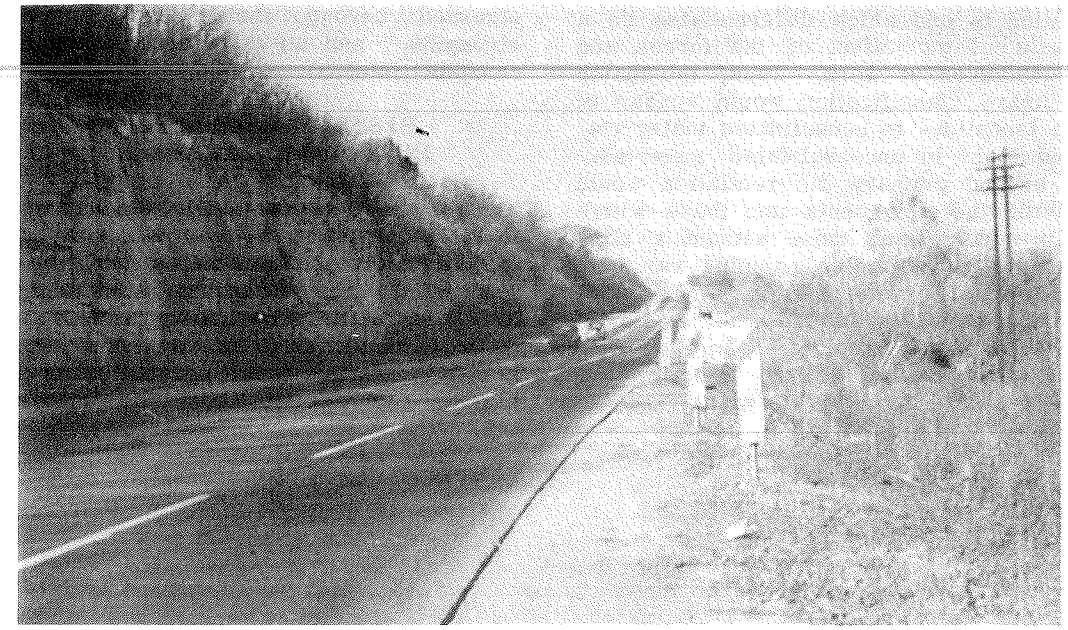


Figure 4. The slide involved in the pictured location is at the right. The problem was solved by shifting the roadway into the stable bedrock at the left of the picture.

mechanics of the movement. The cause of a landslide is often argumentative even after all the available facts have been determined. In many cases, one cause or another may have been the straw that broke the camel's back. Of more importance than the cause, is the realization that increased stability will result by eliminating or minimizing the effect of any contributing factor, particularly that of the effect of the force of gravity.

3. The works of man can measurably accelerate or decelerate the rate of movement of a landslide toward the topographic bottom of the area. Landslides are recognized by geomorphologists as being a major landforming process. The most permanent solutions to control the mass movement will be those of a type that permanently (from a geologic viewpoint) assist nature's resistance.

4. Failure occurs in the soil when the slip-plane is at the contact with the underlying stable bedrock. This observation is valid for all of the instances studied in West Virginia, and was mentioned by Forbes (14) as having been noted in California. Thus, the shear characteristics of the soil at the slip-surface become of primary interest (Fig. 5).

5. For a given landslide problem there is more than one method of correction that can be successfully applied. A common misconception that should be clearly dispelled is that for a given landslide there is one and only one solution. The inference that is undoubtedly intended is that for any given landslide, one method is the most desirable from a consideration of economics, appearance, construction problems, etc.

6. The decision as to the corrective action to be used for a given highway landslide problem is eventually reduced to a problem of economics. This is a statement of an obvious fact, but it is too often subjugated to other considerations. An example that illustrates the point in question would be the case of retaining walls. A wall can be designed sufficiently large to withstand any given landslide. However, a wall design that will be successful may be outside a reasonable range of the economics for a given landslide.

7. Water is a contributing factor in practically all landslides, particularly those involving unconsolidated materials.

Aside from the force of gravity, no factor is more generally present as a contributing factor. The damaging action results from the added weight to the mass, the reduction of shear characteristics of the soil and underlying bedrock (14). Some investigators also state that water produces a lubricating action on the slip-plane. This latter would appear to be a rather unlikely explanation, at least insofar as the mechanics of lubrication are generally accepted.

8. The force of gravity is the sole contributing factor that is common to all landslides. The most obvious basis for a rational analysis is the fact that the force of gravity is the source of all forces tending to cause movement. Until these forces are understood and evaluated, empirical methods are the only available approach.

9. In all mass movements, and just prior to movement, the reactions tending to resist movement are for all practical purposes equal to the forces tending to cause movement. The foregoing statement is an irrefutable fact if the laws of mechanics are valid. Failure to satisfactorily apply a theoretical formula merely means that the method for evaluating the force and the resistance is inadequate. This fact is important since it clearly defines the troublesome features in a rational approach to the mechanics of landslides.

10. The determination of the location of the slip-surface is the most critical factor in the use of a rational or semirational approach. Experience has shown that one of the principal limitations on the use of a theoretical approach is the accurate determination of the location of the slip-surface. The problem is involved in both a theoretical office approach and in field examinations. The latter problems are largely due to the lack of a reliable tool that will rapidly, accurately and inexpensively produce the desired subsurface data.

CLASSIFICATION OF CORRECTIVE MEASURES

In order to clarify the analysis, a

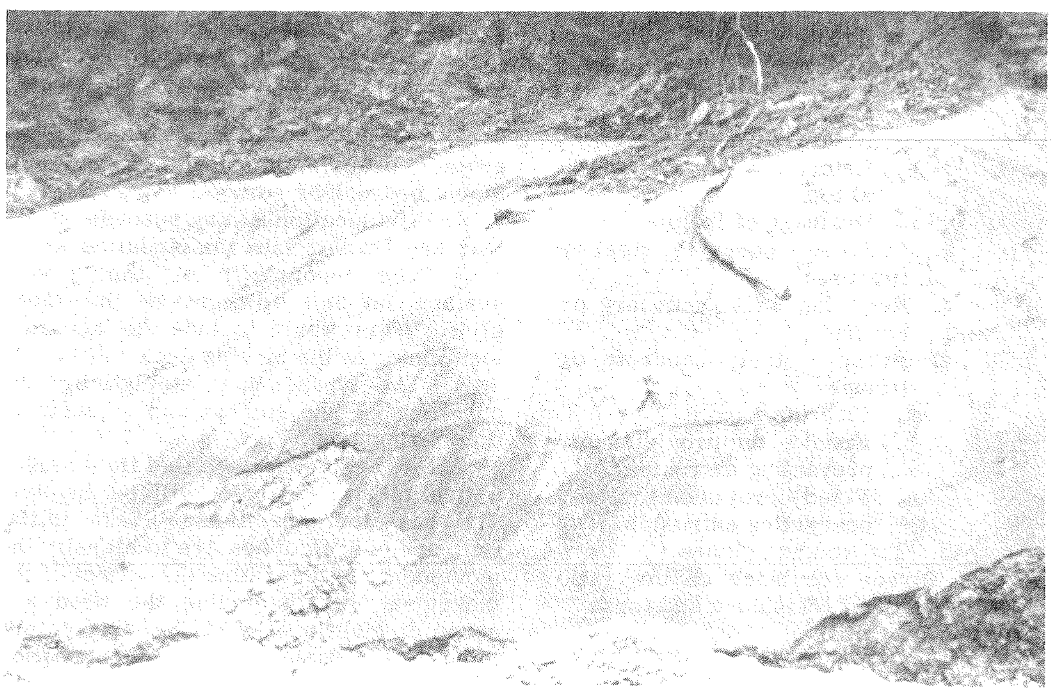


Figure 5. The slip-plane developed approximately 1 in. above a layer of stable shale. The scar at the left of the picture developed as the thin layer of clay dried and cracked.

classification was suggested for various corrective measures commonly used in highway landslide problems. It will be recalled that the fundamental difference lies in whether the method involves elimination or control. The following is a detailed classification of the most common corrective measures currently in use. The basis of the classification is the similarity of the analyses within a given group. More details on the methods are given in Appendixes B, C, and D.

- I. Elimination methods
  - A. Relocation of structure - complete
  - B. Removal of the landslide
    - 1. Entire
    - 2. Partial at toe
  - C. Bridging
  - D. Cementation of loose material - entire
- II. Control methods
  - A. Retaining devices
    - 1. Buttresses
      - a. Rock
      - b. Cementation of loose material at toe
      - c. Chemical treatment - flocculation - at toe
      - d. Excavate, drain and backfill at toe
      - e. Relocation - raise grade at toe
      - f. Drainage of the toe
    - 2. Cribbing - concrete, steel or timber
    - 3. Retaining wall - masonry or concrete
    - 4. Piling - steel, concrete or timber
      - a. Floating
      - b. Fixed - no provision for preventing extrusion
      - c. Fixed - provision for preventing extrusion
    - 5. Tie-rodding slopes
  - B. Direct rebalance of the ratio between resistance and force
    - 1. Drainage
      - a. Surface
        - (1) Reshaping landslide surface
        - (2) Slope treatment
      - b. Subsurface (French drain type)
      - c. Jacked-in-place or drilled -in-place pipe
      - d. Tunnelling
      - e. Blasting
      - f. Sealing joint planes and open fissures
    - 2. Removal of material - partially at top
    - 3. Lightweight fill
    - 4. Relocation - lower grade at top
    - 5. Excavate, drain, and backfill - entire
    - 6. Chemical treatment - flocculation - entire

PRELIMINARY ANALYSIS OF A LANDSLIDE

The foregoing is a lengthy list of methods that have been used successfully in controlling or avoiding landslides. Ladd (3) suggested most of those that appear in the classification. The complete list of possibilities should be considered for each landslide at the start of the analysis.

Four factors are required before one can obtain an understanding of the mechanics of the stability of a landslide. These are:

1. The type, character, and topographic description of the underlying, stable bedrock or soil.
2. The location of any seepage strata that are leading into the landslide area.
3. The topography of the ground surface on and adjacent to the landslide. This would include the accurate locationing of the moving area.
4. The types, characteristics, and condition of the soil in and adjacent to the moving area.

Before beginning a detailed field study, a preliminary analysis will be helpful. The principal objectives of these initial field and office studies are to classify the movement, to determine the extent of the movement, to determine the need and scope of additional study, and to determine the probable methods of correction that will be feasible.

Fortunately for the highway engineer, numerous landslides can be handled by elimination methods, i.e., the landslide can be avoided or removed. In such

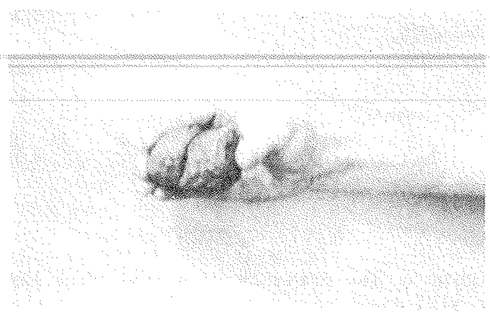


Figure 6. Drilling will occasionally produce excellent evidence of the location of the slip-plane.

cases, a rapid estimate of the costs involved will show clearly the relative economics and general desirability of an elimination method. For those landslides that cannot be typed as one to be eliminated, an estimate is necessary as to what types of control methods are within reason. With experience, it will become increasingly easier to estimate the corrective methods that will be most economical and otherwise desirable. A study of the appendixes that follow will give some indication of the most desirable set of conditions for the various types of corrective measures. The advantage to this initial estimate lies in the savings that can be realized in future field and office analyses.

FIELD STUDY

Where the situation permits, the field study should extend over several months and, in some cases, years. Unfortunately, many highway problems will require an early decision, and extreme effort will be required to delay action until even a superficial analysis can be made. A study that extends over several months differs primarily from a short study in that continuous observations are made of the direction and the extent of the movement, and of the fluctuation of the ground-water table.

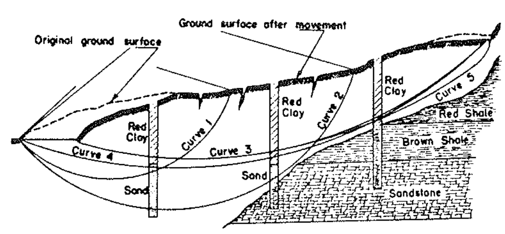
The details to be obtained from the field study will depend upon whether a complete analysis has been deemed necessary. For instance, for certain types of landslides and retaining devices, only the foundation conditions of the retaining device will be needed. As a general rule, however, if a stability analysis is necessary, it will be desirable to obtain the complete information indicated in the preliminary analysis.

In obtaining data concerning the subsurface conditions within the moving mass, various types of drilling as well as geophysical surveys have been used. The most important data to be obtained from this subsurface work are: (1) evidence of the location of the slip-surface (Fig. 6); (2) the condition of the soil as to moisture, density, and structure (for future shear tests); and (3) information that indicates direction and type of movement.

STABILITY ANALYSIS

The following stability analysis is a composite of numerous methods that appear in the literature, and is proposed for use in all landslides involving unconsolidated material. It should be pointed out, however, that applicability of the stability computations to flow and creep movements will require more study, particularly with regard to the location of the potential slip surface. However, by increasing the over-all stability (as indicated by a stability analysis), the actual tendency for flow movement should be lessened.

It is relatively easy to select a corrective measure that will produce a beneficial effect on the landslide area. The purpose of the following analysis is to estimate the degree of stability produced by a given method. In addition, the relative merits and costs of several



Curves 1,2,3,4,and 5 represent potential slip planes.

Figure 7. Slide that developed when the toe was cut. Core-drilling located underlying bed-rock. Curves 1 and 2 were established by theoretical formulae. Curves 3, 4, and 5 were adjusted due to layers of underlying stable material.

methods are studied. It is assumed that the resistance to movement equals the force causing movement at the instant of failure. Formulas developed for use in the theoretical soil mechanics are used in the evaluation. Since all of the corrective measures which are considered are analyzed by the same method, the same relative stability should be obtained. The major point of concern is whether the analysis produces an over-design or occasionally an under-design.

Stability analyses of landslides have been applied in two principal ways. If the shear characteristics of the soil are determined, it is possible to estimate the safety factor of the slope. A second procedure is the determination of the average cohesion, or c of the soil at the slip-surface. With the latter method, laboratory tests are not used to determine the shear characteristics of the soil. In either method, it is most desirable to evaluate the landslide under the conditions which existed before the most recent movement. After the determination of the safety factor or the estimation of the shear characteristics of the soil mass, sufficient data are available to estimate the influence of the corrective action.

The method used in West Virginia consists of the procedure involving the estimate of the average c and the following discussion deals primarily with this type analysis. The first step in the stability

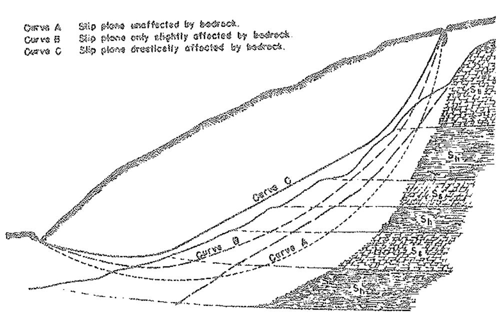


Figure 8. With homogeneous soil, the slip-plane would tend to develop along Curve A. If the area is underlaid by bedrock (as shown in the shaded area) the slip-surface would tend to be as indicated by Curve B. If the bed-rock lies as shown in the solid line, the slip-plane will lie approximately in the position of Curve C.

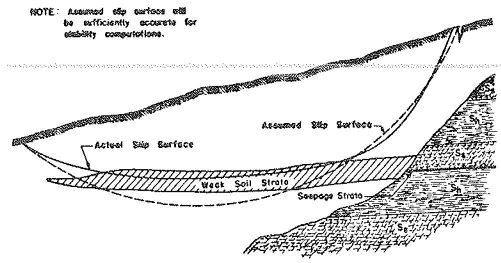


Figure 9. The presence of a weak soil layer will tend to produce a failure within its limits. On occasions, the actual slip-surface will be as close to the theoretical position indicated, and the circle can be used in the computations.

computations is to prepare typical cross-sections parallel to the direction of movement (Fig. 7). The sections should be continued above and below the landslide. On these sections should be plotted all drill information, results of laboratory soil tests, data concerning seepage strata, location of underlying bedrock, surface cracks, structures, and any information considered descriptive of the slide movement. The ground lines both before and after recent movements are very desirable. If the before-movement ground surface is not known, a reasonable estimate will be helpful. The most dangerous sections should then be selected for the initial study. This section will generally be near the middle of the slide, will have the greatest over-all slope (from toe to top), and the greatest mass of unconsolidated material.

The next step is the most troublesome, and perhaps the most vital. The slip-plane must be drawn in its most probable location. The top and bottom of the slide are generally easily identified, but the intermediate portion will call for careful interpretation of the drill data. Observations throughout the past years have led soil engineers to the conclusion that slopes in homogeneous soils fail along surfaces that can be approximated by a circle (in a two-dimensional analysis). Having the top and bottom of the landslide, two points near or on the slip-surface are known. The third, and controlling, point must be estimated. Theoretical formulas (5) suggest a method for the initial approximation. These formulas are for slopes without sur-

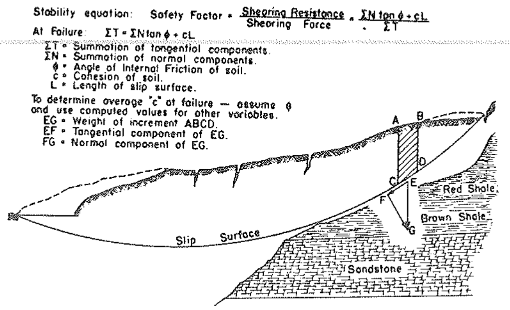


Figure 10. Solution by graphical integration. The total area is divided into increments of the same width as ABCD. Weight of the soil is computed and graphically resolved into its tangential and normal components.

charge and for homogenous materials. However, for the initial approximation, the formulas will be of assistance. The presence of an underlying, firm layer may effect a change in the location of the slip-surface. The change might result in a circle tangent to the layer, two circles connected by a third circle, or two circles connected by a straight line (Fig. 8). The shape of the slip-surface will also be affected by the presence of weak layers (Fig. 9). Taylor (8) has suggested that a circle that approximates a series of curves will be sufficiently accurate.

The drill data will indicate the presence of underlying bedrock or stable soil layers that are in a position to affect the slip-surface. In addition, layers of particularly weak soil can be identified. If there is a question as to the position of the slip-surface, a complete design should be made for each possibility, and the slip-surface that produces the most conservative result should be used.

When the landslide is extensive, slip-planes must be checked for various points up and down the slope (Fig. 7), in addition to the over-all stability. In some cases, several slip-planes will appear reasonable. Each of these should be checked as outlined in the following.

With a reasonable estimate as to the location of the slip-surface, the cross-section of the landslide should be divided into increments, parallel to the direction of movement. Referring to Figure

Figure 11. Photograph of roadway in Kanawha County near Charleston, West Virginia. Note break at right edge of picture.

10, ABCD is a typical increment. The width of the increment is dependent upon the irregularities of the ground surface. Generally, an increment width of 10 to 30 ft. will produce results well within the accuracy of the remainder of the analysis. The weight of the soil in the increment is computed, keeping in mind that the section is assumed to be 1 ft. in width (perpendicular to direction of movement). The weight should be computed for both the original ground surface and the ground surface after movement.

The weight may then be represented by a vector, i. e., a scaled length representing the weight (Line EG). Graphical resolution of this force is accomplished by drawing a line tangent to the centerpoint of the segment of the slip-surface (Line EF). Another line is drawn perpendicular to the tangent at the midpoint of the slip-surface (Line FG).

The intersection of the two lines defines their length. The parallel force is the shear, and the perpendicular force is termed the normal. The resolution of the forces is accomplished for each increment of weight, and the sums of the shear forces (ΣT) and the normal forces (ΣN) are computed.

The forces tending to hold the soil mass in place are (1) the frictional components of the normal forces and (2) the cohesion c of the soil. The forces tending to cause movement are those of shear, seepage, and hydrostatic pressures. There is a diversity of opinion as to the validity of neglecting these latter two forces. Under certain conditions, the hydrostatic forces can be very significant, particularly in cases where cohesionless layers or pockets are present. The effect of the hydrostatic pressure is to reduce the normal forces, and in cohesive soils with a low φ value, the change may be insignificant. The seepage forces tend to decrease the normal force as well as to increase the shearing force and the result is significant in the opinion of Taylor (8). In the initial stability analysis, that follows, hydrostatic and seepage forces are neglected, except in their combined effect at

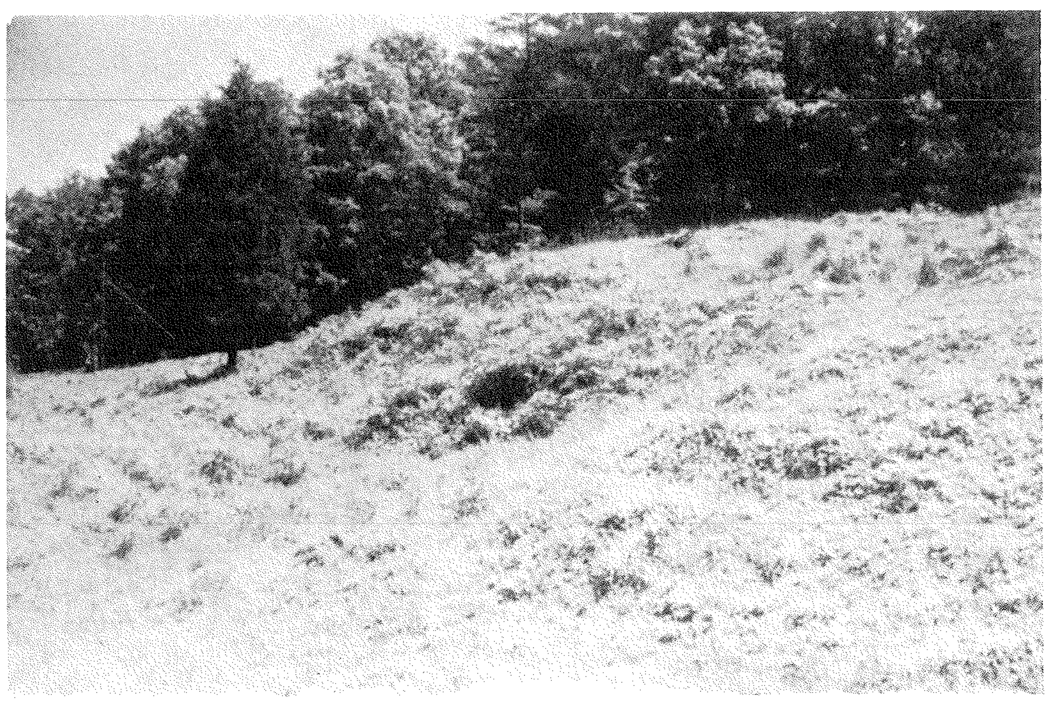


Figure 12. Same slide as that in Figure 11. The toe of the movement is in the middle of the picture.

the time of failure.

A formula that has been proposed for estimating the stability of a slope is the following:

$$\text{Safety factor} = \frac{\Sigma N \tan \phi + cL}{\Sigma T} \qquad (1)$$

where ΣN = the summation of the normal forces in pounds
ΣT = the summation of the shear forces in pounds
ϕ = the angle of internal friction
c = cohesion in pounds per foot
L = length of the slip-surface in feet

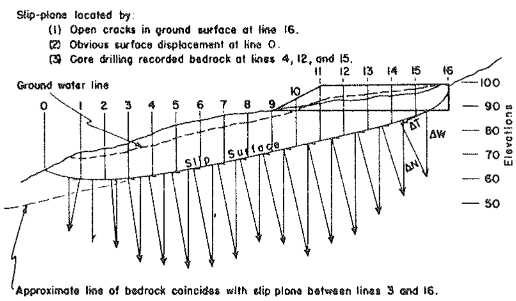


Figure 13. Cross-section of the slide pictured in Figures 11 and 12. The slide has been divided into increments and the computation of ΣT and ΣN is given in Table 1.

Assuming the landslide is at the point of equilibrium between movement and stability (safety factor = 1.0), the following form of the equation is useful:

Shearing force = shearing resistance
or

$$\Sigma T = \Sigma N \tan \phi + cL \qquad (2)$$

It will be noted that the left side of Equation 2 represents the shearing forces causing movement, and the right side is the shearing resistance to movement.

Thus far, the method for obtaining T and ΣN have been indicated. The values of ϕ and c can be determined by shear or unconfined compression tests in the laboratory if desired. Except in rare instances, the laboratory values will not produce a value of 1.0 for the safety factor (Equation 1). This will be true due to irregularities in the soil, to the difficulties in obtaining undisturbed samples, to the problems of laboratory technique,

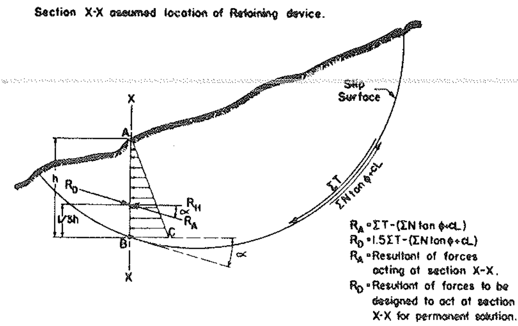


Figure 14. Method for computing the forces acting at a given point in a slide. The value of T and N is determined for the area desired (from X-X to top in this sketch). The difference between the forces causing movement (ΣT) and the resistance to movement ($\Sigma N \tan \phi + cL$) is designated as R_A. The resistance that will produce a safety factor of 1.5 is designated as R_D.

and probably to the effect of hydrostatic and seepage forces. Slopes in nature have been known to be stable even though the safety factor was computed as 0.75. This latter figure would indicate that the shearing resistance was only 75 percent of the shearing force. If the safety factor for a stable slope is less than 1.0, or if greater than 1.0 for an unstable slope, it appears certain that some factor has been disregarded. Numerous examinations have been made of landslide areas, and the computed safety factor was greater than 1.0. Indications were that such computations were based on conditions after the movement. Quite obviously, an area that has moved to a temporarily stable position will show a higher safety factor than 1.0 in its new position. It would appear to be practically impossible to measure the conditions that exist at time of failure. However, if the starting point for the analysis is the ground line prior to movement and Equation 2 is used, the effects of these troublesome variables are accounted for as a part of ϕ or c.

For the following analysis of the stability an estimate is made of the value of ϕ. From Equation 2 it is then possible to compute the average c value needed to obtain an equality between the shearing forces and the shearing resistance. If possible, computations should be carried out for the ground surface con-

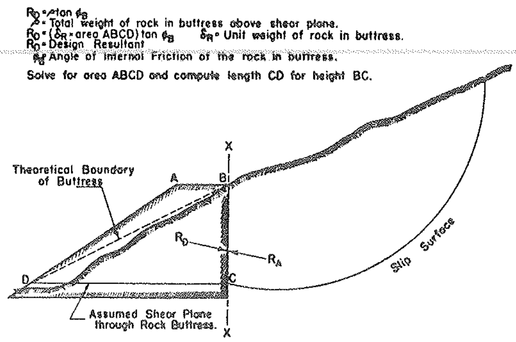


Figure 15. Resistance offered by a rock buttress. The design resultant indicated in Figure 14 must be produced by the shearing resistance of the rock. If the base of the buttress is not bed-rock, a possible shear failure under the buttress must be investigated.

ditions that existed prior to recent movement. From these calculations, the average c value is determined for use in the estimate of the effect of the various corrective measures. The assumption of the ϕ should not lead to serious difficulties. While the c value is very susceptible to varying conditions, the ϕ is relatively constant for a given material. Some investigators have recommended the assumption of $\phi = 0$ for saturated clay soils. This would lead to a more conservative design, since the resistance offered by the normal forces would not be included.

At times it will be necessary to get the range of values for the average c. This will result due to the possibility of various slip-surfaces, and to a range of ϕ values.

APPLICATION OF RESULTS

At the conclusion of the stability analysis the next step is to estimate the change in the safety factor (or in the ratio represented by Equation 2) that is affected by various corrective measures. The definition of the permanency of a correction was made on the basis of the life of the structure. The importance of a differentiation is in the estimate of the economics involved, i. e., whether or not the structure may have to be replaced. Therefore, to be a permanent correction, the safety factor should be increased by 0. 5. This increase can be accomplished by increasing the resistance or decreasing the force. The type of correction governs which of the two (or both) should be changed. Increasing resistance is illustrated in Figure 14 for a retaining device. Unless a significant change can be made in the safety factor, the method is not likely to be helpful on a permanent basis. The use of a corrective action that produces a change of less than 0. 5 in the safety factor must be classed as a calculated risk or an expedient. On the basis of Equation 2, a permanent correction should result in the shearing resistance being 1. 5 times as great as the shearing force for a permanent correction. If the ratio is less than 1. 5, the solution should be considered as an expedient.

The principal difficulty in the follow-up of the stability analysis is the estimate of (1) the additional resistance or (2) the reduction in force that is derived from a specific correction. For elimination methods there are, of course, no problems. Recommended procedures to be used for the control measures are included in the Appendixes C and D. The results thus obtained should not be classed as anything more than an estimate. The degree of accuracy is dependent upon many variables thus far not too well evaluated. In lieu of no other quantitative method, however, the values will be helpful and on the conservative side. In Figure 15, the resistance offered by a rock buttress is illustrated.

In the method involving an estimate of c, it will be interesting to note the relative sizes of the corrections required by applying the upper and lower limits of the range of c values. In many instances, there will be a rather insignificant change in the size of the corrective action needed. For example, a range of 10 deg. in the value of ϕ, made a difference of only 8 percent in the size of a rock buttress. (See Appendix C).

For a given landslide if more than one corrective measure has been indicated as a permanent solution, the final step is an estimate of the costs involved. The decision as to the corrective measure to be employed will be made on the basis of economy, appearance, effect of the change on driver-safety, or by such other means as established as the policy of the organization concerned.

SUMMARY AND CONCLUSIONS

(1) For highway engineers, the basis for the classification of landslides should be on the mechanics of the movement rather than on cause and effect.

(2) For a given highway-landslide problem there are numerous solutions that can be satisfactorily applied, and the problem can be reduced to a problem in economics.

(3) While the detrimental effect of water has been repeatedly emphasized, the fact that has not been sufficiently emphasized is that the force of gravity is always present as a contributing factor.

(4) By classifying the types of corrective measures in common use, it is possible to clarify the method of analysis of a given landslide.

(5) A preliminary analysis of a landslide should lead to an estimate of the types of corrections to be used. This should reduce the cost of investigating some problems.

(6) The field work should produce all possible data on the location of the slip-surface. The critical factor in the office analysis is the accuracy of the delineation of the slip-surface.

(7) At the moment just before failure the force tending to cause movement is equal to the resistance to movement. The problem is to determine these forces.

(8) The analyses of a landslide should be governed by the basic principle of obtaining a more stable slope than existed prior to failure. At the present time, the best method for estimating quantitatively the relative stability is the formula:

$$\Sigma T = \Sigma N \tan \phi + cL \qquad (2)$$

(9) The forces acting against a retaining device can be estimated as can the resistance offered by the retaining device.

(10) The beneficial effect of any corrective action can be estimated in terms of Equation 2.

(11) The procedure suggested may be an over-simplification in its present form. Observations and evaluation of the corrective measures thus far effected will be necessary.

(12) Considerable research work is necessary to better determine the actual shearing forces and shearing resistances at work in a landslide.

(13) Extensive research is needed to determine an inexpensive method for solving highway landslide problems.

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APPENDIX A

TYPICAL STABILITY ANALYSIS

To present an example of the typical computations in a stability analysis, a landslide in Kanawha County, near Charleston, West Virginia, was selected. Two photographs of the area are included as Figures 11 and 12. The area was core-drilled and cross-sections were taken. The slip-surface was relatively easy to locate. The core shown in Figure 6 was taken from this slide. The underlying bedrock and the obvious extent at the top and at the toe limited the possible position of the slip-plane.

After locating the slip-plane, the area was divided into increments. Referring to Figure 13, the slide area was divided into 16 increments. The width of the increments from lines 1 to 15, inclusive, was ten feet. The two end increments were not an established length. These latter two division lines were set so that the weight of one increment (between lines 1 and 2) would not require a resolution of forces.

The areas of the increments were determined by planimeter. The predetermined unit weight of the soil was multiplied by the area and the total weight of the increment computed. It will be recalled that the cross-section is considered to be 1 ft. in width (perpendicular to the direction of the movement).

The weight of each increment was graphically resolved into a component parallel and another perpendicular to the slip-surface at the midpoint of the width (parallel to the movement) of the increment. Table 1 is a summary of the areas, weights, tangentials, and normals for each of the increments. The ΣT and ΣN for the entire slide area are also shown in Table 1.

The length of the slip-surface was determined to be 180 ft. The range of ϕ values that was considered reasonable was 0 to 10 deg. Referring to Equation 2, all of the variables are now available except the cohesion of the soil. The following summarizes the computations involved in determining c.

$$\Sigma T = \Sigma N \tan \phi + cL, \text{ or} \tag{2}$$

$$c = \frac{\Sigma T - \Sigma N \tan \phi}{L} \tag{3}$$

Assuming $\phi = 0$ deg.

$$c = \frac{53,800 - (285,000 \times 0.0)}{180} = 299 \text{ lb. per ft.}$$

TABLE 1

VALUES OF TANGENTIAL AND NORMAL FORCES IN A TYPICAL STABILITY ANALYSIS

	Increment															
	0-1	1-2	2-3	3-4	4-5	5-6	6-7	7-8	8-9	9-10	10-11	11-12	12-13	13-14	14-15	15-16
Increment Area (Sq. Ft.)	104	120	128	160	176	192	192	188	168	179	219	200	193	168	144	124
Increment Weight (Unit Weight = 110 Lbs. per cu. ft.)	11,440	13,200	14,080	17,600	19,350	21,100	21,100	20,650	18,500	19,700	23,800	22,000	21,200	18,500	15,850	13,650
Tangential Force (Lbs.)	-2,100	0	1,000	2,500	3,200	3,500	4,000	3,800	4,000	4,300	5,000	4,800	5,000	4,500	4,500	5,800
Normal Force (Lbs.)	11,100	13,200	14,100	17,100	19,200	21,000	20,800	20,200	18,100	19,200	23,200	21,800	21,000	18,000	15,300	12,500
Intergranular Force (Lbs.)	5,980	6,110	6,240	7,480	8,160	8,850	8,850	9,230	8,290	8,900	9,600	9,350	9,780	8,730	7,480	7,240

$\Sigma T = 53,800$
$\Sigma N = 285,800$
$\Sigma (N - \mu) = 155,530$

Assuming $\phi = 10$ deg.

$$c = \frac{53,800 - (285,800 \times 0.1763)}{180} = 18 \text{ lb. per ft.}$$

The relatively low c value for this silty-clay soil indicates that ϕ is probably smaller than 10 deg., or that there were strong hydrostatic or seepage forces existing at the time of movement.

For the particular slide in question, assume that the ground water lies as shown in Figure 13. The equation which can be used to account for the hydrostatic pressure is as follows:

$$\Sigma T = \Sigma(N - \mu) \tan \phi + cL, \quad \text{or} \tag{4}$$

$$c = \frac{\Sigma T - \Sigma(N - \mu) \tan \phi}{L} \tag{5}$$

Where $\mu = h \gamma_w l$ = water pressure in lb. at the slip-plane
h = depth in feet from ground water line to slip-plane
γ_w = unit weight of water = 62.4 lb. per cu. ft.
l = length of increment in feet along slip-plane

The values for $(N - \mu)$ are listed in Table 1 as intergranular forces.
For $\phi = 0$ deg., there is no change in c due to hydrostatic pressures since

$$\tan \phi = 0.0$$

For $\phi = 10$ deg., the following is indicated:

$$c = \frac{53.800 - (155.530 \times 0.1763)}{180}$$

$$c = 147 \text{ lb. per ft.}$$

APPENDIX B

ELIMINATION METHODS

The five methods included in this classification are:
1. Relocation of structure - complete
2. Removal of landslide
   a. Entire
   b. Partial at toe
3. Bridging
4. Cementation of loose material - entire

The factor common to these methods is the lack of a requirement for a stability analysis. All of the methods depend upon complete avoidance of the landslide or a complete change of the landslide area. The exception may appear to be the partial removal of the landslide at the toe. Ultimately this will lead to near complete removal. In any event, when carried on as an expedient, no stability analysis is used.

As a very general guide, the following is a list of the elimination methods in order of increasing costs.
1. Removal of landslide - partial at toe
2. Relocation of structure - complete
3. Removal of landslide - entire
4. Cementation of loose material - entire
5. Bridging

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I. RELOCATION OF STRUCTURE - COMPLETE

Description - The structure is moved to a location where the foundation is of known stability, either bedrock or stable soil. The grade may or may not be changed, depending upon existing conditions.

Principle Involved - A firm foundation is obtained for the structure.

Best Application - The method is readily applicable to every type of mass-movement. In many cases, the method may prove prohibitive due to excessive cost. The ideal applications are those cases where movements have undermined the structure, and bedrock is located immediately adjacent on the uphill side.

Disadvantages - The cost is usually high if the pavement is of permanent type. Furthermore, the line change may result in poor and unsatisfactory alignment, and finally, the movement is not controlled in event liability is involved.

Method of Analysis - Routine location problem, except particular care should be taken to insure that adequate foundations are available. A complete cost estimate should be made for comparison purposes.

Principle Items in Cost Estimates -
1. Excavation
2. Pavement replacement
3. Right-of-way damages

II. REMOVAL OF THE LANDSLIDE - ENTIRE

Description - All of the slide material is excavated and wasted. This solution applies primarily to movements coming down onto the structure.

Principle Involved - The moving mass that is causing the problem is completely removed.

Best Application - Ideally suited to shallow soil profiles (10 to 20 ft.)and small moving areas (100 to 150 ft. from structure to top of slide). The area above the slide should be stable or worthless, or the question of additional failures should be considered.

Disadvantages - May be too costly for extensive movements. Design care must be taken to insure against undermining the area above, particularly with regard to rockfalls.

Method of Analysis - Normally, the only analyses necessary are for the computations of quantities involved. In cases of questionable stability above the slide area, a stability analysis of the slope above may be required.

Principle Items in Cost Estimate -
1. Excavation
2. Right-of-way damages

III. REMOVAL OF LANDSLIDE - PARTIAL AT TOE

Description - The debris is moved from the area affecting the structure in order to relieve pressure, remove obstacle, etc. Since part of the toe is removed continued movement is inevitable. The method should rarely be used except as an emergency measure. An immediate follow-up with a permanent solution is necessary to prevent future movement.

Principle Involved - The moving mass is excavated so as to permit passage of vehicles, to temporarily relieve pressure against a structure, etc.

Best Application - Very rarely applicable except when movement is down onto structure from above. The method will most often be necessary in instances where the mass has moved against a structure or has blocked a roadway. In instances involving valueless land above, removal of the toe with space provided for future movement may be an economical solution.

Disadvantages - This expedient method does not produce a permanent solution.

Method of Analysis - No analysis is necessary except for quantities involved in a cost estimate. For determining follow-up or permanent correction, a stability analysis of the type required for the permanent solution will be necessary.

Principle Items in Cost Estimate -
1. Excavation
2. Right-of-way damages

IV. CEMENTATION OF LOOSE MATERIAL - ENTIRE

Description - In order to obtain stable material, cement grout is injected into the moving area. This produces a material that has higher shear resistance. In cohesive soils vertical columns are obtained and their effect is that of a system of piling. The same principle is applied when only a portion of the moving mass near the toe is stabilized to produce a buttress.

Principle Involved - The shearing resistance is increased by improving the shear characteristics of the moving mass. In cohesive soils the resisting forces are increased by a transference of load from the moving mass to the underlying stable material.

Best Application - Complete stabilization of the area will not be possible unless the moving mass consists entirely of granular material.

Disadvantages - The principle disadvantage lies in the fact that the method is still experimental and relatively expensive. There is no clear-cut method of estimating the amount of cementing material that will be required. In areas of extensive subsurface seepage, hydrostatic heads may produce flow of entire area unless the pressure is relieved.

Method of Analysis - From a viewpoint of a buttress at the toe, the tone of resistance required can be estimated from a stability analysis. The advantages produced by the cementation will consist of increased shearing characteristics of the soil. The latter values can be estimated by laboratory tests. In instances where hydrostatic heads are involved, the uplift would be a factor in the stability analysis.

The cementation of an entire moving mass is in the category of an elimination method and no analysis is necessary. From a viewpoint of a column action in cohesive soils, the resistance offered by each column can be estimated. Knowing the tons of resistance required for stabilization, it is possible to compute the number of columns required.

Principle Items in Cost Estimate -

1. Equipmental rental
2. Drilling
3. Cement

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Description - The slide area is avoided by a bridge between the two solid extremities of the moving area. Generally, no direct effort is made to control the movement.

Principle Involved - The moving area will not provide a stable foundation for even a part of the roadway or structure. Therefore, firm, unyielding foundations are selected and the area completely bridged.

Best Application - The method is applicable to all types of mass movements. It is particularly suited to steep hillside locations with deep soil profiles, or with bedrock or stable soil at a considerable depth below the desired grade line.

Disadvantages - The main disadvantage is the relatively high cost of the corrective action. In addition, the movement is not controlled in the event liability is involved.

Method of Analysis - Standard bridge design is followed. In most instances, a single span will be desirable due to the lateral thrust that would be applied to a pier constructed within the moving area. Particularly thorough foundation examinations will be necessary to avoid placing the abutments on material that may move in the future.

Principle Items in Cost Estimate -

1. Bridging

APPENDIX C

DESCRIPTION OF CONTROL METHODS - RETAINING DEVICES

The corrective measures included in this classification are:

1. Buttresses
   a. Rock
   b. Cementation of loose material at the toe
   c. Chemical treatment - flocculation - at toe
   d. Excavate, drain, and backfill - at toe
   e. Relocation - raise grade at toe
   f. Drainage of the toe
2. Cribbing - concrete, steel, or timber
3. Retaining wall - masonry or concrete
4. Piling - steel, concrete or timber
   a. Floating
   b. Fixed - no provisions for preventing extrusion
   c. Fixed - provision for preventing extrusion
5. Tie-rodding slopes

Further details are available on the following:
Cementation of loose material - Appendix B
Chemical treatment - flocculation - Appendix D
Excavate, drain, and backfill - Appendix D
Relocation - Appendix B
Drainage - Appendix D

Description - A resistance is placed in the path of the moving mass. The resistance is placed somewhere between the structure, or area to be protected, and the toe of the slide.

Principles Involved - Since all retaining devices produce additional resistance to movement, the benefit derived is resisting force that will be added to the shearing resistance (Equation 2).

Advantages - Retaining devices will often permit correction with the least amount of right-of-way damages. In certain cases, only a part of the landslide is brought under control, and a savings is realized over an attempt to control the entire movement. When the area is exposed to stream erosion, the retainer can be designed as a slope protection device.

Disadvantages - Except for floating piles, most retaining devices represent a relatively expensive solution. In addition, except for cribbing, failure of the method will result in a complete loss of the investment involved in the corrective action.

Method of Analysis - Having completed a stability analysis, the point at which the retainer is to be used is selected. The assumed value of φ and the average c as computed in the stability analysis are used to obtain the summation of the shearing forces and shearing resistances for the area between the location of the retainer and the top of the slide. This is shown diagrammatically in Figure 14. The summation of shearing forces (ΣT) is multiplied by 1.5. This product will represent the summation of the required shearing resistance for a permanent solution. The actual shearing resistance of the soil is subtracted from the required shearing resistance, and the difference is the force that the retainer must be able to resist without failure.

For all retaining devices, the type and location of stable foundations is a critical factor. In the event bedrock is close, the retainer should be anchored into bedrock. In the event the retainer is placed on soil, the foundation must be below the slip-surface (except for the tie-rodding solutions) and a stability analysis must be made assuming that the slip-surface is diverted to a location below the retainer. In this latter case, the primary benefits of the retainer will result from lengthening the slip-surface, and increased normal forces for $\phi > 0$ deg.

The resistance offered by a retaining device will be the minimum value obtained from the following: (1) friction or shear between the base and the underlying bedrock or soil; (2) increase of normal forces on a slip-surface extending beneath the retaining device (for $\phi > 0$ deg.) and the lengthening of the slip-surface with a corresponding increase in total cohesion resistance; and (3) resistance to shear or to overturning of the retaining device.

The only other factor not considered is the bearing capacity of a soil under a retaining wall. However, there will be few cases where the size or dimensions of the retaining wall will be governed by this factor.

In determining the actual resistance offered by the retaining device, each of the applicable factors mentioned above must be investigated. The original design will be based on the factor that usually controls that particular type of device. The design or location of the correction device must be changed until the minimum resistance offered by one of the three factors is approximately equal to the required shearing resistance.

1. Friction between the base and the underlying bedrock or soil - With the exception of piling, one of the sources of resistance for a retaining device is the friction or shear between its base and the underlying bedrock or soil. The formula for estimating this value is:

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$$s = c + p \tan \phi, \quad \text{where} \tag{6}$$

s = shearing resistance of soil in lb.
c = cohesion of the soil at location of slip-plane (lb. per ft.)
p = the weight (direction perpendicular to movement) of the retaining device (lb. per ft.)
ϕ = angle of internal friction between the retaining device and the bedrock (ϕ of the foundation soil).

In cases where the foundation is bedrock, the failure will be at the surface between the bedrock and the base of the retaining device. Thus, for bedrock or granular soil foundations, $c = 0$ and Equation 3 becomes:

$$s = p \tan \phi \tag{7}$$

For bedrock or granular soil, ϕ will range between 25 and 35 deg. A conservative assumption can be made or laboratory tests can be used to determine the value of ϕ.

For cohesive soils within a buttress or under any retaining device, the cohesion will not be the average c determined for the slide itself. The value for c refers to the material beneath or within the retainer and should be obtained from laboratory tests of undisturbed soil samples.

2. Increase of Normal Forces and of Cohesion Forces on a Slip-Surface Extending Beneath the Retaining Device - This factor will apply only to those retaining devices not founded on bedrock. In addition, for piling the cohesion effects apply but not the increase in normal forces. Another qualification, if the device is placed at a higher elevation than the center of gravity of the slide, the load of the retainer will increase the shearing forces on the over-all slope stability. Thus, full advantages of increasing the normal forces and the total cohesion will rarely be realized unless the retaining device is placed at a lower elevation than the center of gravity of the sliding mass. Finally, an increase of the normal forces will not benefit slides in which $\phi = 0$ deg.

The computation of this factor is accomplished by dividing the cross-section into increments similar to those used in the original stability analysis. For the new slip-surface (recalling that the foundations of the device must be placed below the original slip-surface) the summation of the normal forces will be increased and the length will be greater with a corresponding increase of cohesion resistance.

3. Resistance to shear or to overturning of the retaining device - In order to estimate the resistance to shear or to overturning of the retaining device, it is necessary to know the magnitude, distribution, point and direction of application of the forces acting on the retainer. A suggested method for determining these factors is pictured in Figure 15. From the stability analysis, the required shearing resistance can be determined. The horizontal component of the force can be evaluated by graphical resolution. It is then a reasonable assumption that the force decreases uniformly to a value of zero at the ground surface. There is a vertical component of the required tangential force but the vertical force can be neglected unless the retaining device is placed over a steep portion of the slip-plane. This force does change the direction of the resultant. However, it can be assumed that the resultant acts parallel to the slip-surface. See Figure 15.

DETAILS ON INDIVIDUAL METHODS

1. Buttresses - all types - In each instance, the slip-plane should be assumed to be extending through the buttress. For rock and cementation of loose material at the toe, the slip-surface through the buttress can be assumed to be a straight line extension (Fig. 15). The resistance can be computed from Equation 7. The resistance required at this point is the required tangential force obtained in the stability analysis. Theoretically, a rock buttress should be a triangle that is sufficiently large so as to resist the shear at any point. As a practical consideration, however, the top of the buttress is normally built horizontal for 5 to 10 ft. In Figure 15, a line shows the theoretical limits within which the edge of the buttress should fall. The horizontal widths at various levels are defined by the uniform reduction from the maximum at the slip-plane to zero at the ground surface.

For the buttresses involving soil materials, the resistance of the buttress to shear is computed by Equation 6. The ϕ and c of the material in the buttress should be determined by laboratory tests. For the drainage solution, laboratory permeability tests or field well points should be used to determine the feasibility of drainage. Furthermore, the ϕ and c values should be those obtained from laboratory tests on a sample of the soil under the reduced moisture conditions.

Referring to the example used in Appendix A and to Figures 14 and 15, the following is a typical example of the computations for a rock buttress:

For ϕ = 0 deg., c = 299 lb. per ft., L = 144 ft. for buttress at line 3.

ΣT = 54,900 lb. (from line 3 to 16, inclusive)

ΣN = 247,400 lb. (from line 3 to 16, inclusive)

$1.5 \times \Sigma T = 82,350$ lb.

$cL = 144 \times 299 = 43,000$ lb.

$\Sigma N \tan \phi + cL = 0 + 43,000 = 43,000$ lb.

$R_D = 82,350 - 43,000 = 39,350$ lb.

$R_D = (\gamma_R \times \text{Area ABCD}) \tan \phi_B$

γ_R = 100 lb. per cu. ft.

ϕ_B = 30 deg.

$\text{Area ABCD} = \dfrac{39,350}{57.54} = 680$ sq. ft.

For ϕ = 10 deg., c = 18 lb. per ft.

$1.5\ T = 82,350$ lb.

$\Sigma N \tan \phi + cL = (247,400 \times 0.1763) + (18 \times 144) = 46,140$ lb.

$R_D = 82,350 - 46,140 = 36,210$ lb.

$\text{Area ABCD} = \dfrac{36,210}{57.75} = 632$ sq. ft.

From the foregoing, the condition of ϕ = 0 deg. gives an 8 percent more conservative figure than that of ϕ = 10 deg., therefore, design the buttress with at least 680 sq. ft. Assuming that the exposed slope of the buttress is on a 1 1/2 : 1 slope (Horizontal : Vertical), and the backslope is vertical:

$$\text{Bases of buttress} = \frac{\text{Area}}{h} \pm \frac{1.5h}{2}$$

If h = 16 ft.

$$\text{Top width} = \frac{680}{16} - 12 = 30.5 \text{ ft.}$$

$$\text{Base width} = \frac{680}{16} + 12 = 54.5$$

The principle items of cost in a buttress are as follows. Not all of the items will be required in every buttress.

(a) Excavation
(b) Backfill (Rock or Soil)
(c) Admixture (Cement or Chemical)
(d) Drainage Pipe
(e) Drilling (for Admixtures)
(f) Equipment Rental (for Admixtures)

2. Cribbing and Retaining Walls - Use is made of standard design methods for the type of wall under consideration. Cribbing should be considered as a gravity-type wall. The magnitude, point of application, and direction of the stresses against the wall will be as indicated in Figure 14.

3. Piling - In order to be fully effective, the piling should extend one-third of its length below the slip-surface. The following is a formula for resistance to shear of the piles (7):

$$s = \frac{A_p \times f_v}{D}, \quad \text{where} \tag{8}$$

s = shearing resistance offered by a pile, in lb. per inch (in a direction perpendicular to the movement)
A_p = cross-section area of the pile in square inches
f_v = allowable stress in shear for the pile, pounds per square inch
D = center-to-center spacing of the piles in inches

A pile should also be checked for the resistance to the soil shearing along each side of the pile. A formula has been suggested by Hennes (7):

$$s = 2\,chd \tag{9}$$

s = shearing resistance per pile in lb.
c = cohesion of the soil, lb. per sq. ft.
h = height from slip-surface to ground surface in feet
d = diameter of the pile in feet

A sufficient number of piles must be available so that the soil shearing resistance or the sum of the shearing resistance of the piles are equal to or greater than the required resultant of the horizontal forces (Fig. 15). The piling will not be subjected to cantilever action until movement has occurred. Due to partial restraint offered by the surrounding material, it should not be necessary to compute the stability of the piles from a cantilever viewpoint unless there is a possibility of movement of the area below the piling.

Referring to the example used in Appendix A, no experienced engineer or geologist would be likely to recommend piling at the location selected for the buttress (Line 3). The computations verify this opinion:

Assuming a 12 in. diameter, timber pile with a cross-section area of 113.1 sq. in., and ϕ = 0 deg., c = 299 lb. per ft.

$$s = \frac{A_p \times f_v}{D} \qquad (8)$$

f_v = 100 lb. per sq. in.
$s = \frac{39,350}{12}$ lb. per in.

$$D = \frac{113.1 \times 100 \times 12}{39,500} = 3.45 \text{ in.}$$

$$s = 2\,c\,h\,d \qquad (9)$$
$$= 2 \times 299 \times 15 \times 1 = 8970 \text{ lb. per pile}$$

$$\frac{39,350}{8970} = 4.4 \text{ piles per ft.}$$

$$D = \frac{12}{4.4} = 2.7 \text{ in.}$$

For ϕ = 10 deg., c = 18 lb. per sq. ft.

$$s = 2 \times 18 \times 15 \times 1 = 540 \text{ lb. per pile}$$

$$\frac{39,350}{540} = 73 \text{ piles per ft.}$$

$$D = \frac{12}{73} = 0.16 \text{ in.}$$

The obviously low value of 18 pounds per ft. for the cohesion is not a legitimate figure to use unless the material is very fluid. It will be recalled that the average c of 18 lb. per ft. represents the material at the slip-plane. It is not unreasonable to expect a much weaker material at the slip-plane. A more legitimate value for c in Equation 9 is a representative c for the material from the slip-surface to the ground surface at the location of the piles. This could be adequately determined from laboratory tests.

A more reasonable location of piling would be at Line 10 (Fig. 13). The computations follow:

For ϕ = 0 deg., c = 299, 12 in. diameter timber piling

ΣT = 29,600 lb. (lines 10 to 16, inclusive)
L = 66 ft.
1.5 T = 44,400
cL = 19,734
R_D = 24,666
D = 135,800 = 5.5 in.

For average c = 400 lb. per sq. ft. (above slip-plane)

$$s = 2 \times 400 \times 16 \times 1 = 12,800 \text{ lb. per pile}$$

$$\frac{24,600}{12,800} = 1.7 \text{ piles per ft.}$$

$$D = \frac{12}{1.7} = 6.7 \text{ in.}$$

The use of steel or concrete piles would permit wider spacing. The computations would be similar to those for timber piling. However, even a location near the roadway would require very close pile-spacing for a permanent solution.

4. Tie-Rodding Slopes - Resistance will be offered by the piling, cribbing or other retaining device. The remainder of the required resultant must come from the anchorage system. The required resultant (Fig. 15) must be equalled or exceeded by the combined resistance of the retainer and anchorage. The resisting force obtained from the tensile strength of a number of steel bars of a given dimension.

Relative Cost - As a very general guide, the following is a list of the retaining devices in order of increasing costs:

1. Piles - floating
2. Buttress - rock
3. Buttress - excavate, drain and backfill at toe
4. Buttress - relocation - raising grade at toe
5. (a) Buttress - cementation of loose material at toe
   (b) Chemical treatment - flocculation - at toe
6. (a) Cribbing
   (b) Piling - fixed - no provision for preventing extrusion
7. (a) Tie-rodding slopes
   (b) Piling - fixed - provision for preventing extrusion
8. Retaining wall

APPENDIX D

CONTROL METHODS - DIRECT REBALANCE OF RATIO BETWEEN RESISTANCE AND FORCE

The corrective measures included in this classification are:

1. Drainage
   a. Surface
      (1) Reshaping landslide surface
      (2) Slope treatment
   b. Sub-surface (French drain type)
   c. Jacked-in-place or drilled-in-place pipe
   d. Tunnelling
   e. Blasting
   f. Sealing joint planes and open fissures
2. Removal of material - partially at tope
3. Light-weight fill
4. Relocation - lower grade at top
5. Excavate, drain, and backfill - entire
6. Chemical treatment - flocculation - entire

Further details are available on the following:

Relocation - Appendix B
Removal of Material - Appendix C

Chemical Treatment - Flocculation - Appendix C
Excavate, Drain and Backfill - Appendix C
Drainage - Appendix C

Description - The forces that are contributing to the movement are decreased or the natural sources of the resistance to movement are increased. There is no artificial treatment with the exception of chemical treatment.

Principles Involved - The drainage solutions may depend upon the reduction of the shearing forces by the elimination of part of the weight of the moving mass. Drainage may also increase the shearing resistance by increasing c or increasing the intergranular forces (normals) by eliminating hydrostatic pressures. Methods other than drainage either reduce the shearing stresses to a greater extent than the reduction of the normal forces or increase the c value of the soil by increased densities or by treatment of the soil. Chemical treatment may also reduce the water-holding capacity of the soil, which would tend to reduce the shearing forces. Blasting combines the advantages of drainage and the permanent displacement (vertically, upward) of the slip-surface. The slip-plane dispacement by blasting tends to reduce the shearing forces by decreasing the weight of the moving mass, while the beneficial effects of drainage are probably temporary.

Disadvantages - Most of the drainage methods are rather costly, as are excavating, draining and backfilling and chemical treatment. Also, the estimate of the value to be obtained from a drainage solution is extremely difficult. For sealing joint planes, there is a problem of determining whether or not the seepage will develop in another location.

There may be construction problems in installing drainage below the slip-surface in the moving mass. Furthermore, the advantages from drainage of cohesive soil masses may be delayed or may never develop due to low permeability. 129

Method of Analysis - Having completed the basic stability analysis and having the average c value to be used, the reduction in shearing forces is estimated: for the drainage solution by estimating probable reduction of unit weight of the moving mass, and for removal of material at top, relocation by lowering grade at top, and the light-weight fill. The increase of shearing resistance results from the increase of c value for the following: all drainage solutions (except blasting); excavating, draining, and backfilling; and for chemical treatment. There is an increase of normal force due to eliminating hydrostatic pressures for all drainage solutions, and for excavation, drain and backfill.

The method of analyzing for hydrostatic pressures is a complex field problem of measuring existing ground-water levels (or excess hydrostatic pressures) and estimating probable maximum height. In computations, the effect is shown by Equation 5 or Appendix A. If hydrostatic pressures are to be considered, Equation 5 should be used instead of Equation 1 or 2, in the original stability analysis.

The excess hydrostatic pressures will be particularly troublesome in landslides that contain pockets or layers of free-draining material. It is probable that such pressures are also troublesome in areas where water is relatively free to move down the slip-plane.

Terzaghi (4) points out that in impermeable soils, flash pressures may develop due to heavy rains. Such pressures are relieved before a significant change can be brought about in the water table. He, therefore, recommends a form of piezometric tube to observe these phenomena in the field.

It should be emphasized that the effort to check the effect of hydrostatic pressures is necessary in the procedure outlined herein in order to determine the degree of improvement brought about by drainage solutions. The values obtained by Equations 1 and 2 will be misleading from an academic consideration. However, it is assumed that the most serious condition has been accounted for in the computation of the average c. The

TABLE 2

DETAILS FOR LIGHT-WEIGHT FILL

	Increment						
	9-10	10-11	11-12	12-13	13-14	14-15	15-16
Weight of original soil (lb.)	19,700	23,800	22,000	21,200	18,500	15,850	13,650
Increment area (sq. ft.)	25	75	105	105	105	105	116
Increment weight (unit weight = 110 lb. per cu. ft.)	2,750	8,250	11,550	11,550	11,550	11,550	12,700
Weight of soil (lb.)	16,950	15,550	10,450	9,650	6,950	4,300	950
Weight of L.W. fill (unit weight = 40 lb. per cu. ft.)	1,000	3,000	4,200	4,200	4,200	9,200	5,920
Total weight of soil + L.W. fill (lb.)	17,950	18,550	14,650	13,850	11,150	8,500	6,870
Normal force (lb.)	17,500	18,200	14,500	13,750	10,850	8,200	6,300
Tangential force (lb.)	3,920	3,900	3,200	3,260	2,720	2,410	2,920

For area between lines 9 and 16, inclusive, with light-weight fill:
$\Sigma N = 89,300$ lb.
$\Sigma T = 22,330$ lb.

For entire area with light-weight fill:
$\Sigma N = 89,300 + 154,800 = 244,100$ lb.
$\Sigma N = 22,330 + 19,900 = 42,230$ lb.

130 relief of hydrostatic pressure by the installation of a drainage system is not reflected in the stability computations using Equations 1 and 2.

Referring to the example used in Appendix A, for the case of $\phi = 10$ deg., the average c was computed to be 147 lb. per ft., using Equation 5. If a drain were installed at Line 16, below the slip-plane, and in a position to lower the groundwater table so that it coincided with the position of the slip-plane, the following computations indicate the improvement in stability:

$$\text{S. F.} = \frac{\Sigma(N - \mu)\tan\phi + cL}{T} \qquad (5)$$

$$= \frac{(285,800 - 0) \times 0.1763 + (180 \times 147)}{53,800}$$

$$= 1.43$$

Thus, the installation of the drain would increase the safety factor by 0.43, which would be sufficient to be termed a permanent solution.

TABLE 3

DETAILS FOR RELACATION - LOWERING ROAD GRADE AT TOP OF SLIDE

	Increment							
	9-10	10-11	11-12	12-13	13-14	14-15	15-16	Σ
Weight of soil (lb.)	16,950	15,550	10,450	9,650	6,950	4,300	950	
Tangential force (lb.)	3,700	3,270	2,280	2,280	1,690	1,220	405	14,845
Normal force (lb.)	16,500	15,100	10,400	9,550	6,750	4,180	870	63,550

For area between lines 9 and 16, inclusive:
$\Sigma N = 63,550$ lb.
$\Sigma T = 14,845$ lb.

Referring to the same example the value to be obtained from a light-weight fill can be estimated as follows.

Assume that the area between Lines 9 and 16, inclusive, and above the elevation 90.0 is to be removed and replaced with a light-weight material that weighs 40.0 lb. per cu. ft. (unit weight of original soil = 110 lb. per cu. ft.). Table 2 summarizes the change in normal and tangential forces between Lines 9 and 16, inclusive.

Assuming ϕ = 0 deg., c = 299 lb. per ft.

$$\text{S. F.} = \frac{\Sigma N \tan \phi + cL}{\Sigma T} \qquad (1)$$

$$= \frac{(244,100 \times 0) + (299 \times 180)}{42,300}$$

$$= 1.27$$

Assuming ϕ = 10 deg., c = 18 lb. per ft.

$$\text{S. F.} = \frac{(244,100 \times 0.1763) + (18 \times 180)}{42,300}$$

$$= 1.1$$

Thus, the light-weight fill increases the safety factor by 0.1 to 0.27. This would not be sufficient to be considered a permanent correction.

If the grade of the road were lowered to an elevation of 90.0, the following S. F. is obtained (data in Table 3):

Assuming ϕ = 0 deg., c = 299 131

$$\text{S. F.} = \frac{\Sigma N \tan \phi + cL}{ST} \qquad (1)$$

$$= \frac{0 + (299 \times 180)}{34,745} = 1.55$$

Assuming ϕ = 10 deg., c = 18

$$\text{S. F.} = (218,150 \times 0.1763) + (18 \times 180)$$

$$= 1.20$$

Therefore, lowering the grade would fall slightly short of being a permanent solution. The degree of importance to attach to the ϕ = 10 deg. assumption would be the controlling factor.

Relative Cost - As a very general guide, the following is the list of the methods that modify the shearing resistance or shearing force. This list is in order of increasing cost:

1. Surface drainage - reshaping landslide surface
2. Surface drainage - slope treatment
3. Blasting
4. Light-weight fill
5. Removal of material - partially at top
6. Relocation - lowering grade at top
7. Jacked-in-place or dilled-in-place pipe
8. Subsurface (French drain type)
9. Tunnelling
10. Sealing joint planes or open fissures
11. Excavate - drain - backfill - entire
12. Chemical treatment - flocculation - entire

Project Correspondent Jorge Jauregui Canevaro, Director, Servicio Nacional de Caminos, Bolivia (photo courtesy of International Road Federation, Washington, D.C.)

HANDBOOK

ON

LANDSLIDE ANALYSIS AND CORRECTION

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CONTENTS

Chapter 3 SLOPE DESIGN IN BED ROCK CUTS

Field investigation and examination of rock samples should be conducted with a view to determining the resistance of rocks to weathering and the probability of a rock fall as a result of discontinuities in the rock strata. A field examination of outcrops of the bed rock will generally provide some kind of a general answer. It may also indicate the possibilities of irregularities in rock likely to exist subsequent to the blasting required for excavation. If there are such indications, rock falls should be regarded as a potential danger and accordingly the slope angle of the cut should be chosen judiciously. As a rule, a slope should be fitted to the material and not the material to the slope. The concept that all the rocks are brothers under the skin and should be cut on $\frac{1}{4}$ to 1 is faulty. There are all sorts of rocks and what is a good slope for one type in one situation may be an extremely poor slope for a different rock in the same situation. Table 4 is meant as a guide for general use.

In situations demanding high rock cuts, rock fall whether incidental or continuous, threatens road safety considerably. The approach to the rock fall problem should be one of trying to restrain the rocks from falling by resorting to methods such as benching of slopes, flattening of slopes, pinning down loose rocks, covering the area with wire mesh etc. (See Chapter 6). Although certain designs are taking into consideration the fall out zones, they do not keep the rocks off highways. Recent researches have shown that the above relationship is arbitrary and unrealistic. The correct approach must recognise

(a) Minimum width of fall out zone as determined by the frequency of stones making impact at a maximum distance from the base of the cliff.

(b) Destroying the angular velocity of falling stones as a result of impact.

(c) Physical and chemical properties of rock.

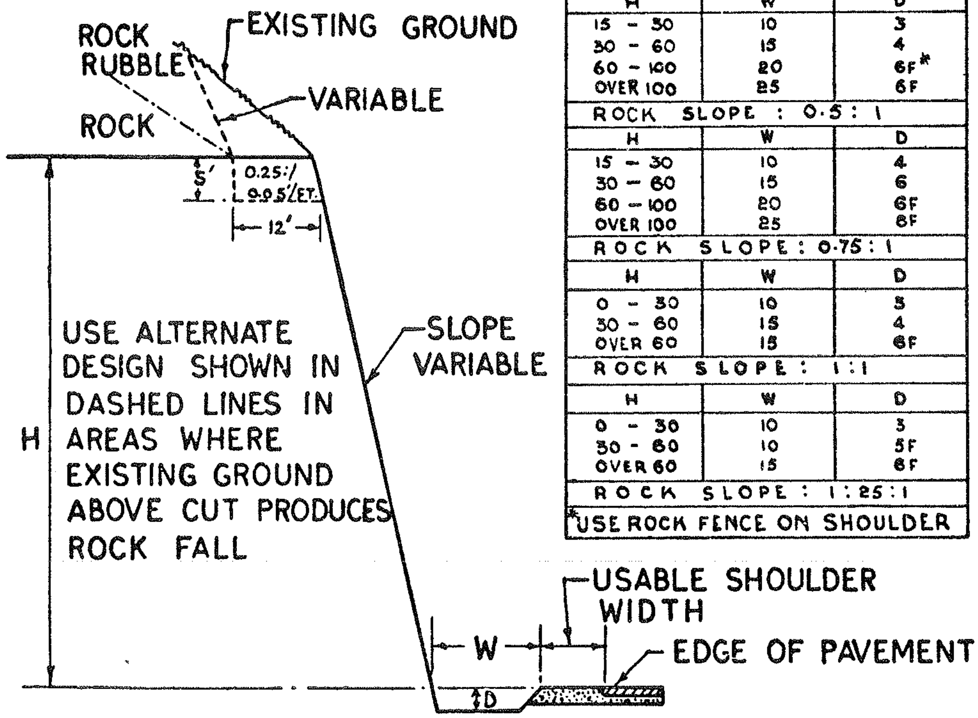
As a result of study on hard basaltic rocks of all sizes, a relationship between width of fall out versus height and angle of slopes has been established, Table 5. Since softer rocks

TABLE 4. **AVERAGE SLOPE VALUES FOR BED ROCK EXCAVATIONS**

			horizontal : vertical
(I) Igneous			
granite, trap, basalt and lave	...	...	1/4 : 1 to 1/2 : 1
(II) Sedimentary			
massive sandstone and limestone	...	...	1/4 : 1 to 1/2 : 1
interbedded sandstones, shales and limestone	...	...	1/2 : 1 to 3/4 : 1
massive clay stone and silt stone	...	...	3/4 : 1 to 1 : 1
(III) Metamorphic			
gneiss, schist and marble	...	...	1/4 : 1 to 1/2 : 1
slate	...	...	1/2 : 1 to 3/4 : 1

TABLE 5. **RELATIONSHIP OF VARIABLES IN DITCH DESIGN FOR ROCK FALL AREAS**

ROCK SLOPE NEAR VERTICAL		
H	W	D
15 – 30	10	3
30 – 60	15	4
OVER 60	20	4
ROCK SLOPE : 0·25:1 TO 0·3 :1		
H	W	D
15 – 30	10	3
30 – 60	15	4
60 – 100	20	6F*
OVER 100	25	6F
ROCK SLOPE : 0·5 : 1		
H	W	D
15 – 30	10	4
30 – 60	15	6
60 – 100	20	6F
OVER 100	25	8F
ROCK SLOPE : 0·75 : 1		
H	W	D
0 – 30	10	3
30 – 60	15	4
OVER 60	15	6F
ROCK SLOPE : 1:1		
H	W	D
0 – 30	10	3
30 – 60	10	5F
OVER 60	15	6F
ROCK SLOPE : 1:25:1		
*USE ROCK FENCE ON SHOULDER		



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are less resilient and do hardly have rebound characteristics, they are less violent on impact. Also, with 4:1 off-shoulder slopes, many rocks roll as much as 80 or 90 ft. from the base of the cliff. Nearly all rocks make their initial impact within a 20 ft. fall out zone. Eventually, the attempt to dampen the rolling velocity or to deflect the falling stones back towards the cliff seems to be an obvious solution in reducing the width of fall out necessary to contain the rock fall.

It has been observed on the basis of field investigations that the use of a 'rock fence' to decelerate a rolling stone and to retard its angular velocity would prove really effective in controlling rock falls. The installation is mounted on the slope above the ditch line or on the back side of guard rail posts on the shoulder of the road, Fig. 19. The fence, nearly 6 ft. high, is suspended like a curtain, from a cable. The cable, in turn, pulls on a compressive spring to absorb the shocks of the rocks and is supported by fence posts at intervals of 50 ft,

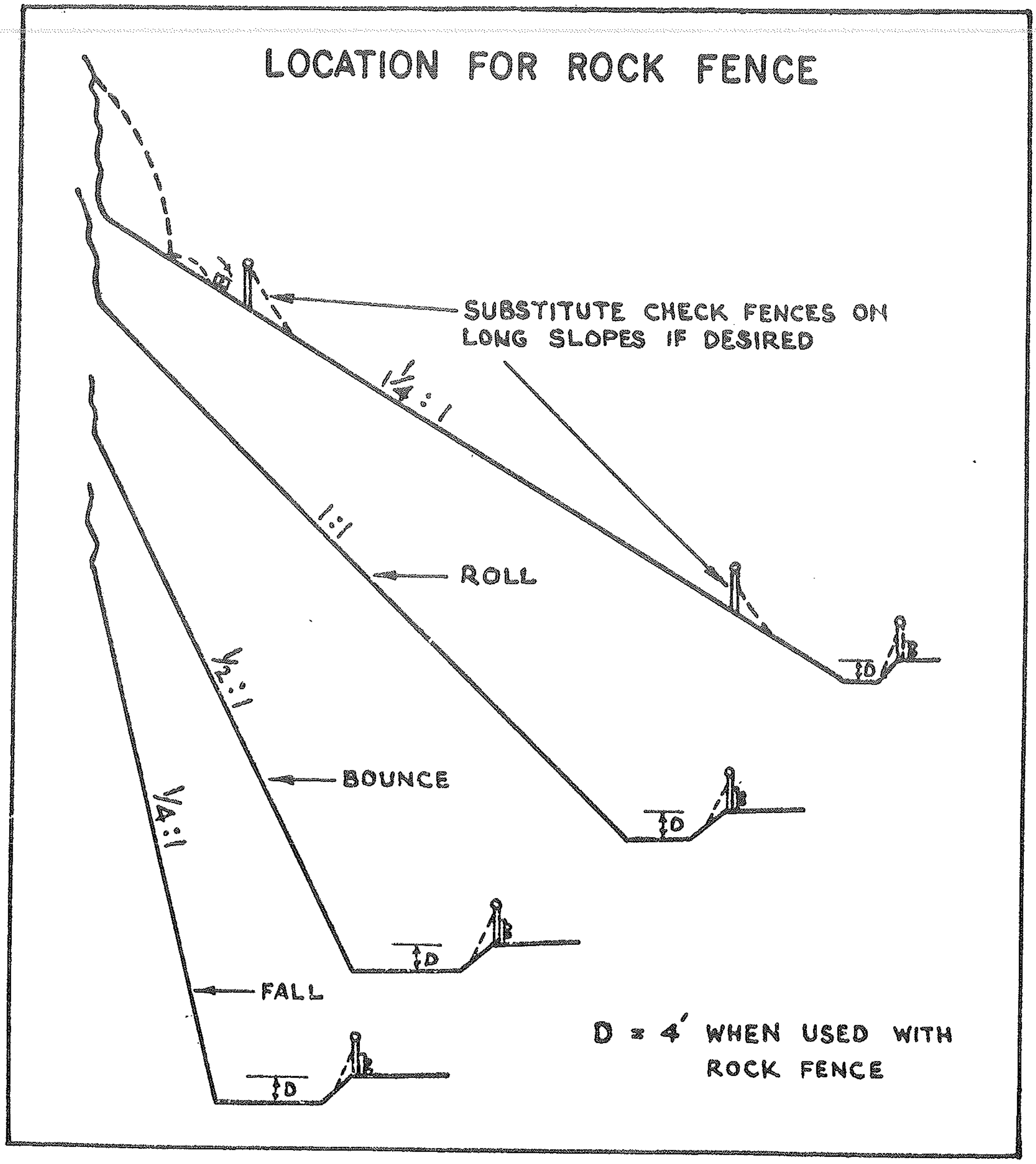

Fig. 19 Position of installation of rock fence used either as check fence on slope or as protection on highway. Though ditch design for $\frac{1}{4}$:1 and $\frac{1}{2}$:1 for a 100 ft. high slope is same, yet rock $\frac{1}{2}$:1 slope must cover a greater distance before encountering same fence

In rock fall areas, gentle off-shoulder slopes provide the ramps for stones to come up on the highway and therefore steeper shoulder design should be preferred in place of the gentle one; for what was thought to be a feature contributing to the overall safety of highway has now been proved to be a detriment and a hazard in rock fall areas. Alternatively, after the factors of weathering and discontinuities have been analysed, an appropriate choice

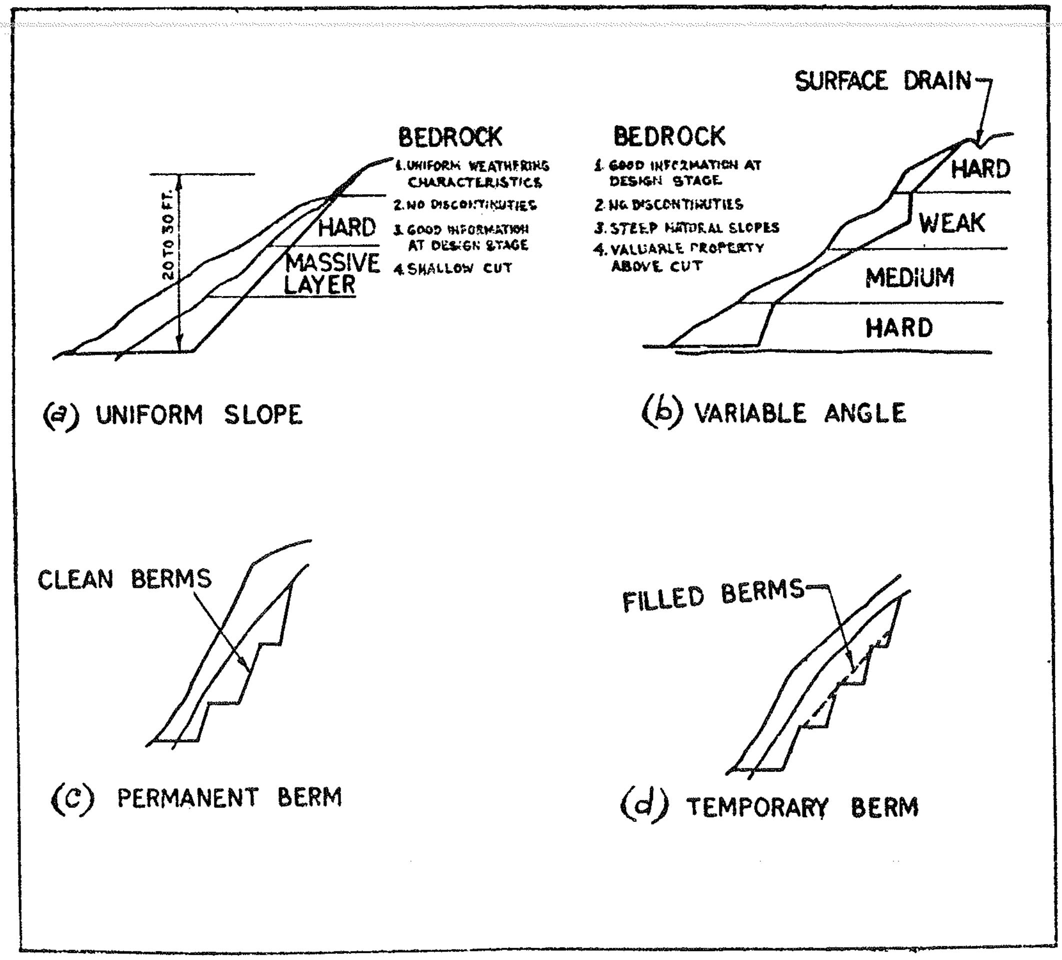

Fig. 20 Types of bedrock cut slopes

of the type of slope to be adopted should be made, viz., uniform, variable angle, permanent berm or temporary berm (see Fig. 20). If discontinuities exist, one of the berm solutions should be adopted in order to intercept the anticipated fragments from a rock fall. If no discontinuities exist, a suitable type of slope has to be selected on the basis of the resistance of rock to weathering, vertical height of the cut and economic considerations.

Table 6 indicates the conditions of suitability for each type of slope.

The design of benches should include important considerations such as longitudinal and transverse slopes, minimum width and location with respect to strong or weak strata. For weak strata, the longitudinal slope should be parallel to the grade line from both constructional and aesthetic considerations, (see Fig. 21 (a)). For hard rock stratum, however, longitudinal slope may be permitted to follow the surface of the layer as shown in Fig. 21 (b).

TABLE 6. **CONDITIONS OF SUITABILITY OF EACH TYPE OF SLOPE**

Type of slope	Suitability	Remarks
(I) Uniform slope	(1) Vertical height of cut is less than 20 to 25 ft. (2) Uniform cross section is involved and uniform weathering is anticipated. (3) No discontinuities exist.	For heights greater than 50 ft., uniform slopes should not be used. Between heights 25 and 50 ft., uniform slopes may be used if the rock strata exhibit uniform weathering characteristics and contain no discontinuities.
(II) Variable Angle	(1) No discontinuities exist in the rock strata. (2) Differential weathering is anticipated. (3) Fairly accurate data on the weathering properties of individual rock stratum are available.	When a deep cut of 100 ft. or more is to be made with steep natural slopes, an elaborate investigation may be justified and variable angle slopes may be adopted without berms. Variable angle design for use in cuts exceeding a height of 30-50 ft., the general data given in Table 4, should be regarded as not very reliable.
(III) Berms	(1) Recommended for stratified sedimentary deposits. (2) Where discontinuities exist in rock stratum. (3) Where rocks are liable to disintegrate on exposure for any reasonable slope angle. (4) Where data on the type, thickness, elevation and the weathering properties of the bed rock are meagre.	Although berms increase the initial cost of construction, subsequent maintenance expenditures are cut down, thereby more than offsetting the increased cost of capital investment.
(a) Permanent berm	Where the angle which the rock will tend to weather is known.	Debris on the berms is not desirable.
(b) Temporary berm	(1) Where very little data on discontinuities and weathering are available. (2) Under all conditions for which permanent berm may be a solution. (3) Where economy is a major consideration.	(1) Debris on the berm is desired so that part of the slope is protected from exposure to air or moisture. (2) The appearance of the slope (irregular and disfigured) gives an impression of an incomplete design.
(IV) Bench		
(a) Single bench	Where good weathering data is available.	(1) It serves as a protection against fall of fragments over the road. Berm should be cleaned off periodically. (2) Should not be used if discontinuities are located more than 30 ft. above the berm.
(b) Multiple bench	(1) Where rock stratum contains discontinuities. (2) Where weathering data is either uncertain or inadequate.	For general guidance, heights and widths are included in Table 7 and Fig. 35 of Chapter 6.

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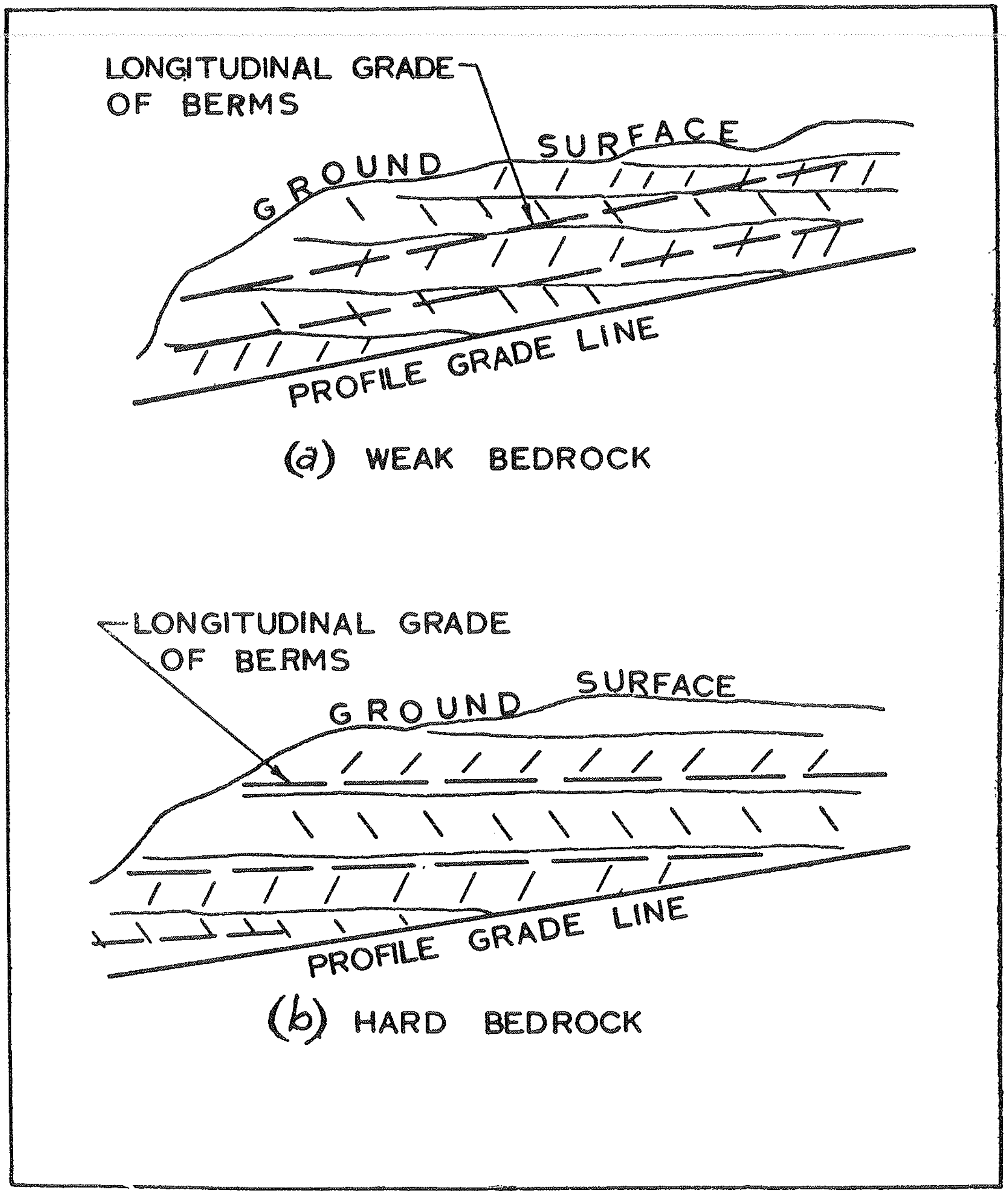

Fig. 21 Longitudinal grade for bedrock berms

At places where soft material is encountered and thin interbedded strata require blasting, the minimum recommended width of bench should be 20 ft. The first bench should be made still wider, say 25 to 35 ft., to receive anticipated fragments of rock fall. As an interceptor for the weathered material, construction of a ~~small~~ bench of 5 ~~to~~ 10 ft. is also desirable at the road level.

Chapter 4 CHARACTERISTIC FEATURES OF LANDSLIDES PECULIAR TO DIFFERENT SOIL TYPES

4.1 SLIDES IN DETRITUS

'Detritus' is a term referring to a loose accumulation of relatively intact pieces of rock intermingled with completely weathered ones. Usually, detritus constitutes a blanket soil covering a gentle rock slope for a thickness of about 20 or 30 ft. Whenever such a soil occurs at the foot of a steep rock cliff, it is called "talus". Whereas in the dry or permanently drained state, detritus can be stable with standard slopes, they are rendered totally unstable as soon as they get saturated, so much so that they start to flow even on gentle slopes. Particularly, detritus slides on gentle slopes occur only in material composed of weak, brittle or partly decomposed fragments of chlorite-talc or mica-schists. The Nashri slide along the Jammu Srinagar Road, by and large, fits the pattern of detritus slides, although the clays partake of the characteristics of stiff fissured clays in isolated patches.

4.2 SLIDES IN SAND

Sand of any kind, permanently situated above the water table can be considered to be generally stable in cuts with standard slopes. Dense and medium sands located below the water table are also equally stable but slides occur mostly in loose saturated sands. Since slides occur only if a sand is very loose, the tendency towards sliding can be cut down by increasing the density of sand. The densification can be achieved by several different means, such as by pile driving or by vibrations induced by explosive charges.

4.3 SLIDES IN LOESS

Loess is a wind-blown deposit having an effective grain size between 0.01 and 0.05 mm. It can easily be mistaken for a normal silt. Although it consists generally of silt-sized particles, the soil grains are cemented together by calcium carbonate or clay. The soil is invariably characterised by its ability to stand vertical or almost vertical slopes, when dry. When a loess deposit has not had an opportunity to get completely saturated, it proves to be a stable soil, but for the fact that it is readily attacked by erosion. On the other hand, when the loess deposit gets submerged or saturated, it is rendered very unstable, acquiring the characteristics of a thick viscous liquid. This happens because leaching removes most of the cementing material, thereby transforming the loess into a cohesionless and unstable mass.

The sudden subsidence of a road in some situations can be attributed to the collapse of the structure of a loess or a loess-like soil upon saturation.

4.4 SLIDES IN STIFF CLAY

By and large, stiff clays are generally characterised by a net work of hair cracks, Fig. 22. When the surface of weakness subdivides the clay into small fragments, one inch

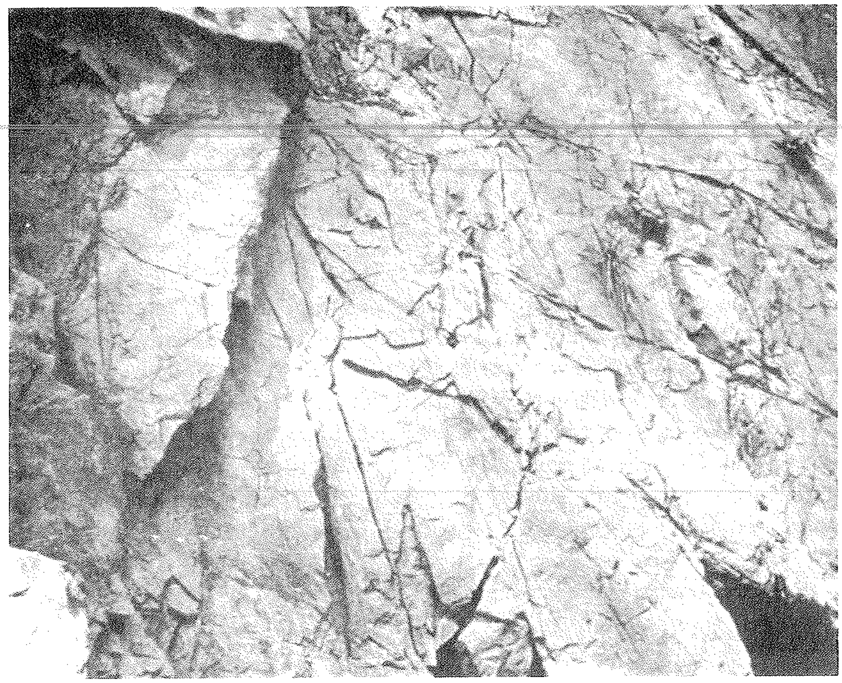

Fig. 22 Cracks in stiff fissured clay

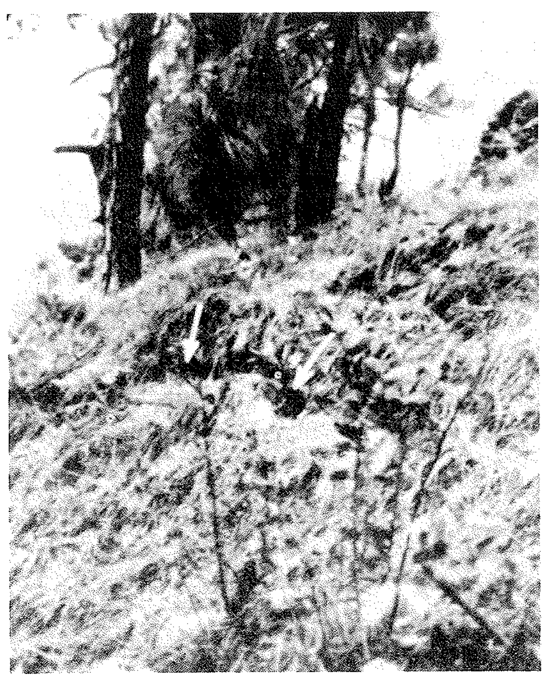

Fig. 23 Development of tension cracks

or less in size, it is likely that the slope may become unstable during the cut or shortly thereafter. On the other hand, if the spacing of the joint is greater, it is likely that the failure may not occur until many years after construction.

The stiff clays have gone through a process of consolidation. The overburden pressure which caused this consolidation had subsequently got removed, resulting in what is called "over-consolidation". When cuts are made in such over-consolidated clays, such as shales, the soil swells due to relief of stress. Consequently, some of the fissures in the clay open out. Water then enters and softens the clay adjoining the fissures. Unequal or non-uniform swelling produces new fissures and the process of disintegration continues with the result that sliding occurs as soon as the shearing strength of the clay becomes too small to resist the force of gravity. Most slides of this nature occur along toe circles, involving a relatively shallow body of soil, because the shearing resistance of the clay increases rapidly with increasing depth below the exposed surface. Certain clay shales in U.P. and Kashmir represent some of the most troublesome types of stiff fissured clays encountered in India. Fig. 23 shows the development of tension cracks.

4.5 SLIDES IN CLAY CONTAINING LENSES OR POCKETS OF WATER BEARING SAND OR SILT

If a cut is made in a clay slope which is underlain by a layer or by even a seam of sand or silt, it is conceivable that excessive pore pressures may develop in the pervious layers, when the entire slope may fail by spreading out horizontally along the pervious stratum. Invariably, failures in such a soil occur suddenly. It is also characteristic of this type of failure that a gentle clay slope which may have been stable for many decades or centuries, moves out suddenly along a broad front. At the same time, the soil in front of the slide flows for a considerable distance beyond the toe of the original slope.

Catastrophic slides of this type are known to occur very suddenly and the surface of failure often tends to lie mostly at or near the interface between the clay layer and the sand parting.

Consider a section through a valley located above a thick stratum of soft clay merging into the sand as shown in Fig. 24 (a). Assuming that some thin horizontal layers of fine sand or coarse silt, such as S-S, occur within the clay stratum, it can be seen that the pore water in layer S-S communicates with the water in the large body of sand. Ad and Be respectively, represent the positions of the water table in the sand during a dry and during an exceptionally wet season, respectively. Ab and Be, respectively, represent the corresponding piezometric levels for pore water in S-S and ab represents a cut made into the clay to a depth H. Every horizontal section beneath the cut including that through S-S is acted on by shearing stresses, because of the tendency of overlying clay to settle vertically and spread horizontally under the influence of self weight. If the porewater pressure in the layer S-S is low, corresponding to the piezometric line Ab, the shearing resistance along S-S is likely to be considerably greater than the sum of the shearing stresses. When this is true, the stability of the slope merely depends upon the cohesion value of clay. For any slope angle less than 53 degrees, the critical height Hc of the slope is given by

$$Hc = \frac{5.52c}{\gamma}$$

where γ is the unit weight of the clay. It may be noted that if a firm base underlies the clay stratum at a shallow depth below the bottom of the excavation, the critical height

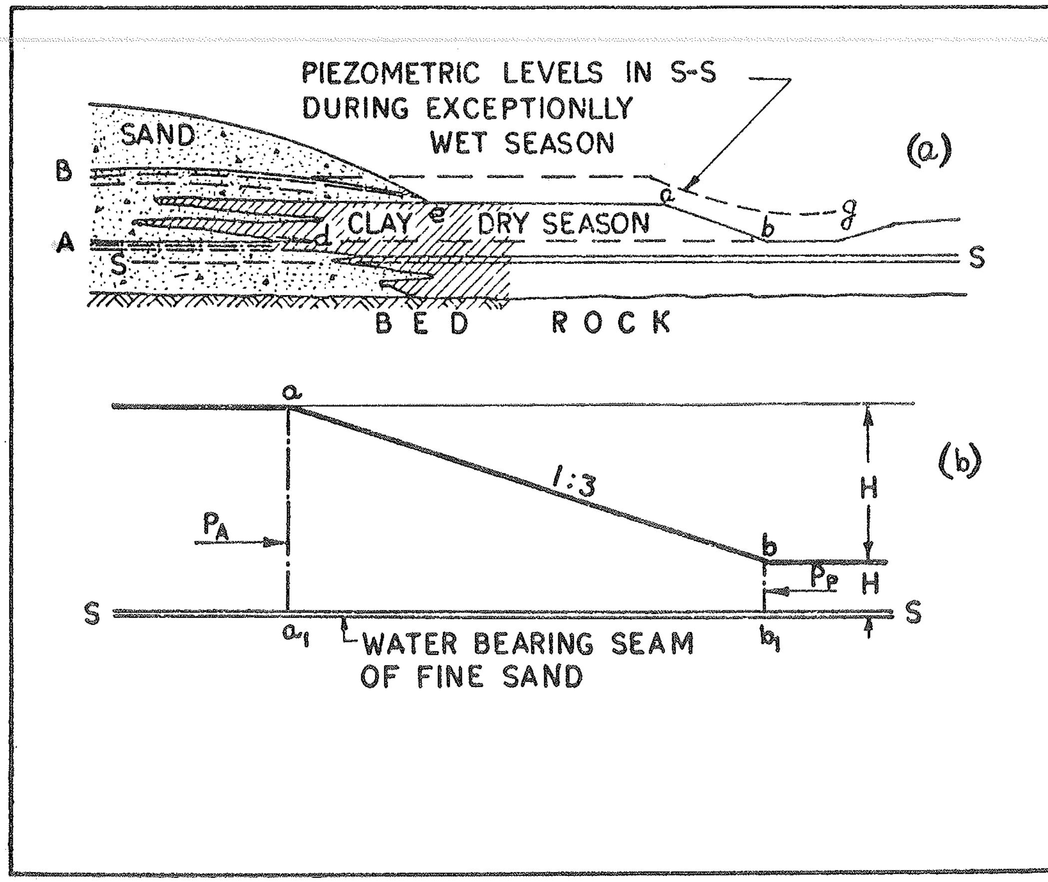

Fig. 24 (a) Geological conditions involving danger of slope failure by spreading
(b) Diagram of forces which act on soil beneath slope ab

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is even greater and it increases with decreasing slope angle upto a value of $\frac{9c}{\gamma}$ for slope angles of 20 degrees.

Now, if the piezometric levels for the stratum S-S rises to the position indicated by line Bg, due to exceptionally heavy rainfall or floods, the pore water pressure u_w would increase. Since the layer S-S is made up of a cohesionless soil, its shearing resistance is determined by the equation

$S=(p-u_w) \tan \phi$

Hence, the shearing resistance on any horizontal section through the layer decreases with an increase in piezometric levels. When the available shearing resistance decreases to the value of shearing stresses, the slope above S-S fails by spreading, inspite of the fact that it may have an adequate factor of safety with respect to rotational failure.

When the pore water pressure in the layer equals p, the shearing resistance along S-S becomes zero. The implications of this condition are illustrated by Fig. 24(b).

If P_A and P_P denote the active and passive earth pressures on the vertical sections aa_1 and bb_1 respectively and if the shearing resistance on $a_1 b_1$ is zero, the slope will be on the verge of failure when P_A equals P_P.

The active earth pressure on aa_1 is

$P_A = 1/2\,\gamma\,(H+H_1)^2 - 2c(H+H_1)$, and the passive earth pressure on bb_1 is $P_P = 1/2\,\gamma\,H^2{}_1 + 2\,cH_1$ if the slope is on the verge of failure, $P_A = P_P$. If $P_A = P_P$, then it can be shown that $H = Hc = \frac{4c}{\gamma}$ which value is more or less equal to the critical heights for a vertical slope, given by the expression $\frac{3.85c}{\gamma}$. It can, therefore, be seen that the development of excessive pore pressures in seam S-S reduces the critical height of the slope to a little more than the critical height for a vertical slope, regardless of what the actual slope angle may be.

The danger of instability by this mode of failure cannot be, therefore, very much reduced or eliminated merely by flattening the slopes of the cut. The correction has to be in the direction of preventing the possibility of the sand or silt layer becoming the seat of excess hydrostatic pressure. This can sometimes be accomplished by the use of relief wells or by vertical drains or by horizontal drains.

4.6 For the choice of the most appropriate corrective measure, for such landslide types, see Chapter 6.

Chapter 5 FIELD AND LABORATORY INVESTIGATIONS

Many minor landslides probably will deserve no more than a cursory inspection by the landslide specialist who will be interested in merely classifying the slide before prescribing the corrective measures. Very often, landslides are contributed by more than one factor and it becomes necessary to assess the importance of various factors involved. Any attempt towards the attenuation of even one of the several factors, can provide the needed stability to the slope. For actual methods of analysis concerning stability of slope, please see Chapter 2.

The purpose of the field investigation is very often to map a landslide, to obtain and record in a graphic form such data as can be observed in the field, to facilitate significant inferences to be drawn relative to the cause, mechanism or potentiality of movement, behaviour of slopes in the past and in the present. The map scale should be decided upon after studying the extent and economic importance of the slide under consideration. A scale of 5 to 10 ft. to an inch may be suitable for small slides whereas a slide covering several hundreds of acres may have to be mapped on a scale of 50 to 100 ft. to an inch. However, important structures of slide area might be selected for mapping on a much larger and a more revealing scale.

The preparation of contour map may be often desirable to map topography of the area. The absolute necessity of such a map and choice of suitable contour interval is generally left to the judgement of the engineer.

The field methods employed in mapping the slide area are selected according to the importance of the work and the degree of accuracy required. Plan and profile illustrations are necessary to exhibit complete graphic portrayal of the slide. The accuracy of this mapping becomes more important if continuing ground movement exists or is anticipated.

Regular surveying methods may be adopted for preparing contour maps and for determining the positions of reference points within and outside the slide area.

It should be borne in mind that survey and mapping of affected ground alone is not adequate, but its relationship to the associated terrain etc., is especially important. The area to be included in mapping is a function of topographic features but the following may serve as thumb-rules to help the judgement of the investigator.

(1) Along or parallel to contours, the map should extend about twice the width of the slide on both sides of the slide area.

(2) Across the contours, or up and down the slide, the minimum distance that the map should illustrate is the first sharp break in the slope above the slide crown and below the slide toe.

The mapping should indicate all sources of water in the area adjacent to the slide area.

Complete description of thickness, classification of soil, texture and structure, relative permeability etc., should be recorded. The bed rock should be mapped according to normal geologic methods to show the type of rock and its structure, including bedding, cleavage, joints, folds, faults or any other geologic features of significance to the slide.

The purpose of subsurface investigation is to determine the physical, geologic and mineralogic characteristics of the slide, the location of the surface and depth of the slide.

Bore holes, test pits and test trenches are most commonly used means of subsurface exploration. The shear tests, standard penetration tests or soundings may be particularly useful for determining approximate locations of weak strata, and by inference, the slip surface. For homogeneous, deep, soft clay deposits, the vane shear tests should be particularly helpful in obtaining reliable values for shearing resistance.

Recovery of undisturbed samples whenever necessary should be made by a rotary drilling rig. The nature of laboratory tests generally depend on the problem in hand. These tests may be routine identification tests such as field moisture content, Atterberg limits, mechanical analysis etc., and/or shear tests such as the direct shear, the triaxial or the unconfined compression tests, or they may involve mineralogic or weathering tests.

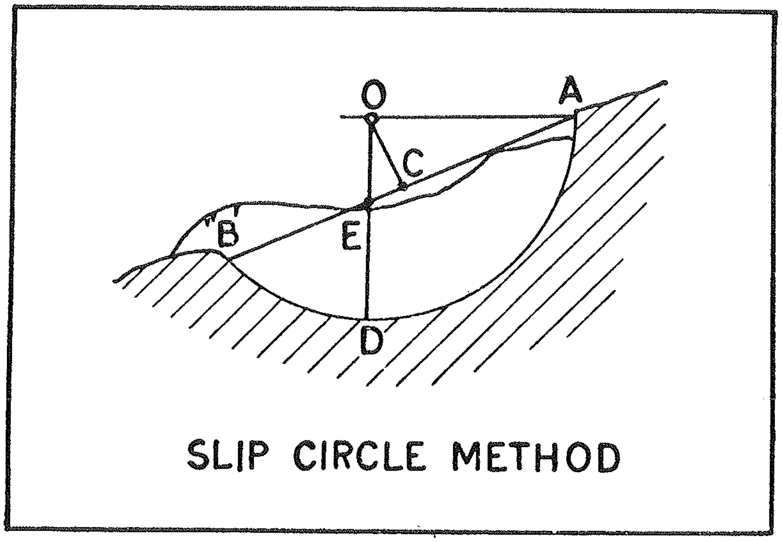

Fig. 25(a) Slip circle method for estimating depth of a slump slide

Determination of the Depth of Rotational Slides

A reasonable estimate of the maximum depth of a slide is often helpful in deciding upon the depth of subsurface exploration and to get an idea about the magnitude of the slide etc. Consider a slump block as shown in Fig. 25 (a). Only the field measurements relating to the position of the crown A and the foot B and the profile of the ground between A and B are required. Make a plot of points A and B and the ground line on a graph. Locate point O at the intersection of the perpendicular bisector OC of line AB and the horizontal line OA. With OA as radius describe an arc, defining the maximum depth of the slide material at point D.

Alternatively, if there has been an appreciable offset of some distinct reference points, such as the edge of a pavement, an idea of the position of the centre of rotation and therefore

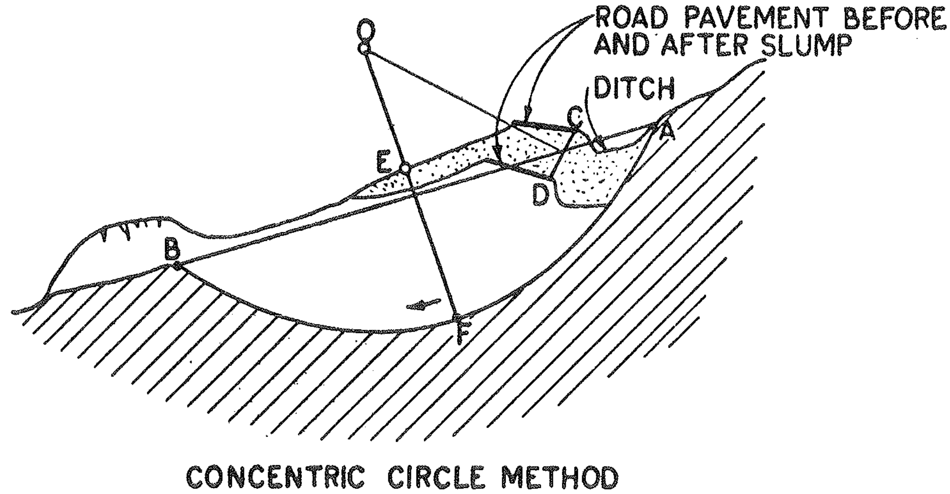

Fig. 25(b) Concentric circle method for estimating depth of a slump slide

the maximum depth of the slide, can be determined as shown in Fig. 25 (b). Positions of points C, D and the crown A can easily be determined. The foot B can also be determined or estimated. Points A, B, C and D are plotted on a graph. Draw lines AB and CD and bisect each line. The bisectors will intersect at point O, which represents the centre of rotation of a unit slump block. By scaling the distance FE directly from the graph, the maximum thickness of the slide is determined.

Any of the above two methods can be applied to a slump made up of several individual blocks. This is done by having the geometry of the lowest block in the series analysed, because invariably it happens that the rupture surfaces of individual slump blocks of a multiple block slide tend to lie tangent to a common shear plane.

Computation of Shearing Resistance from Slide Data

A field study of landslides in a particular area yields valuable information regarding the average value of shear strength of soil, s, in that area. On the basis of on the spot study, the shape of surface of sliding should first be ascertained. The surface of sliding is then replaced by the arc of a circle having radius 'R' and centre 'O' as shown in the Fig. 26.

Referring to figure 26

$$W_1L_1 = W_2L_2 + sR\,\widehat{d_1 e_2}$$

from which

$$s = \frac{W_1 L_1 - W_2L_2}{R\,\widehat{d_1 e_2}}$$

Where W_1 is the weight of slice a k f e which tends to produce failure, and W_2 is the weight of slice k b d_1 which tends to resist it.

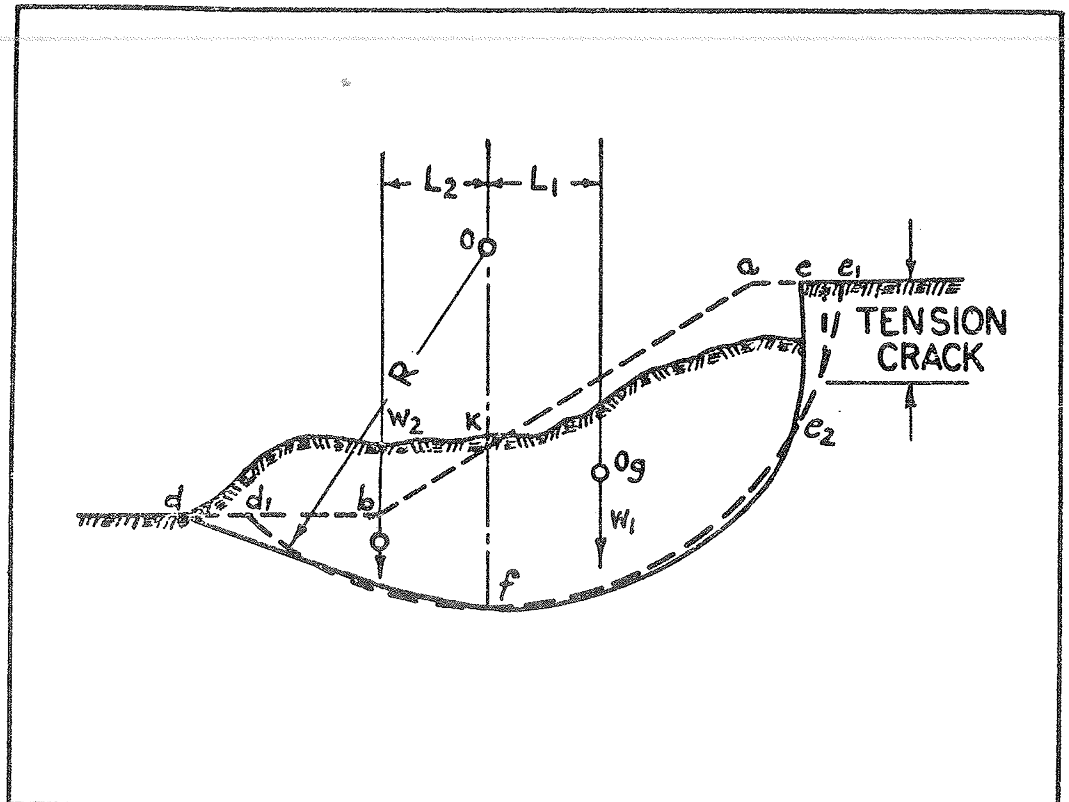

Fig. 26 Deformation associated with slope failure

Significance of Cracks

It is very necessary on the part of the landslide investigator to cultivate the faculty of observing small cracks and displacements in the surface soils and to interpret their significance or meaning. This ability to read into the significance of cracks can add to the understanding of the causes and character of movements, that is very often a prerequisite to correction. For instance, tiny cracks observed at or near a boulder or root of a tree should be regarded as evidence of stretching of the ground surface. Soil creep and stretching of the ground surface are generally regarded as two separate things. 'Stretching' is to be distinguished from 'soil creep' inasmuch as it indicates a comparatively deep seated movement, whereas soil creep is more of a superficial origin. Very often, stretching is observed in granular materials that cannot form or retain minor cracks readily. Invariably, small cracks that surround or emanate from a rigid body, such as a tree-trunk or a root or a boulder in an otherwise homogeneous soil, constitute evidences of stretching. Since tensional forces tend to concentrate at or near rigid boundaries, these cracks are found in association with objects like tree-trunks, roots or boulders. Fig. 27 shows small *en-echelon* cracks which are discernible in the surface soil of the affected area. These cracks generally form long before other signs of rupture manifest. Therefore, they should be regarded as particularly useful tools for the recognition of potential landslides. It can generally be said that in many a

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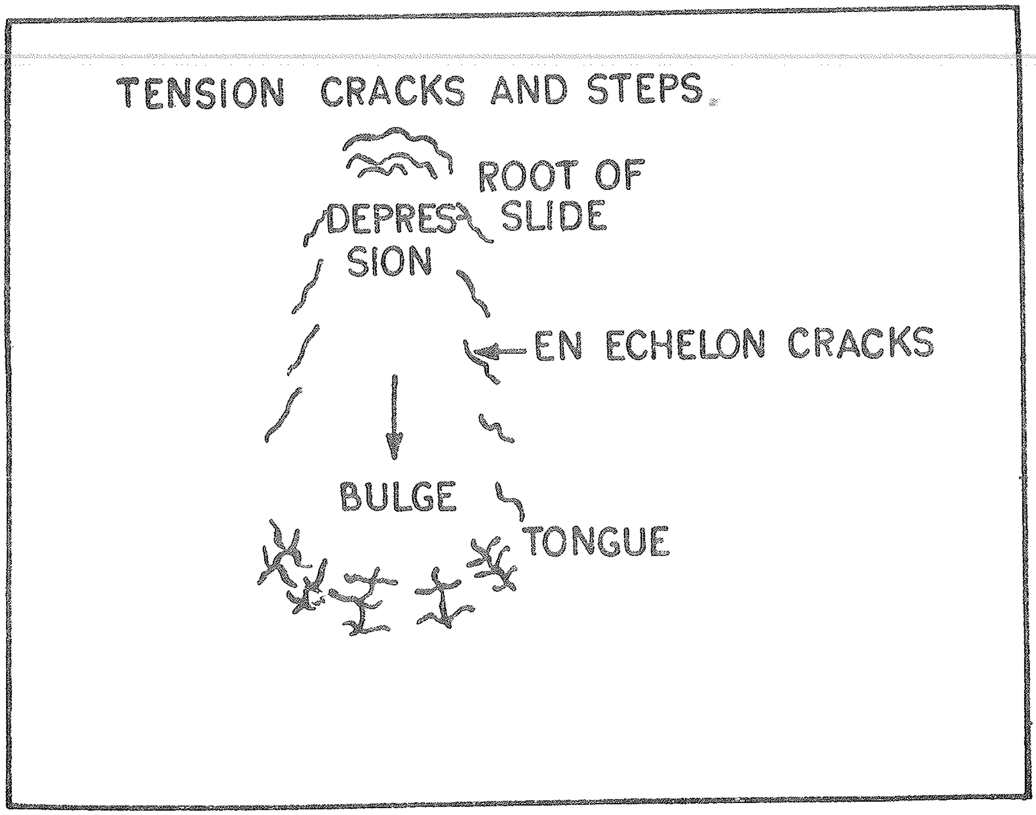

Fig. 27 Plan of typical slide in cohesive material

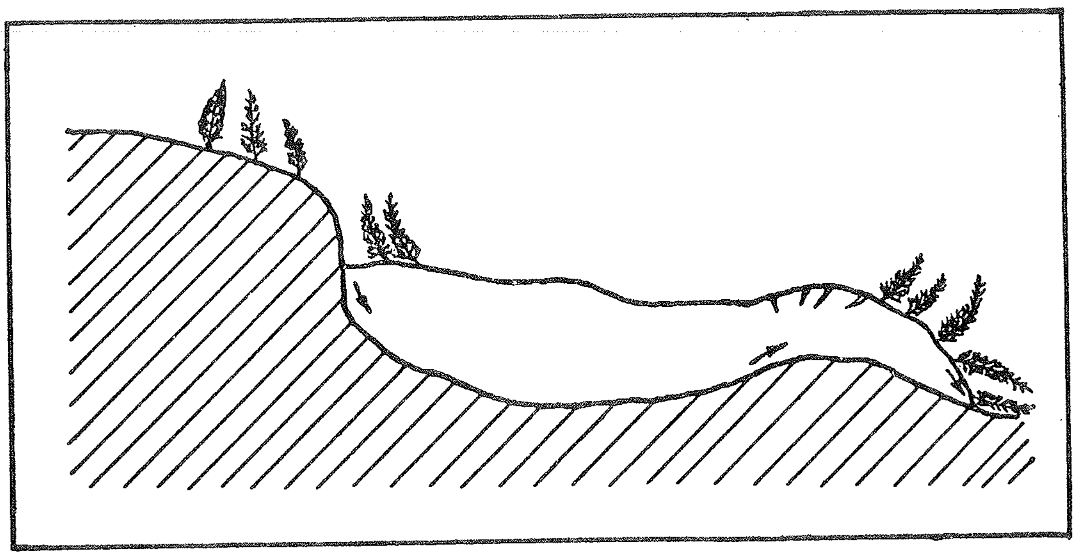

Fig. 28 General orientation of trees on a slump landslide. Because of rotation, the trees on the blocks are bowed uphill, a result of the tree tops tending to grow vertically while the stump portion changes with the rotating land surface

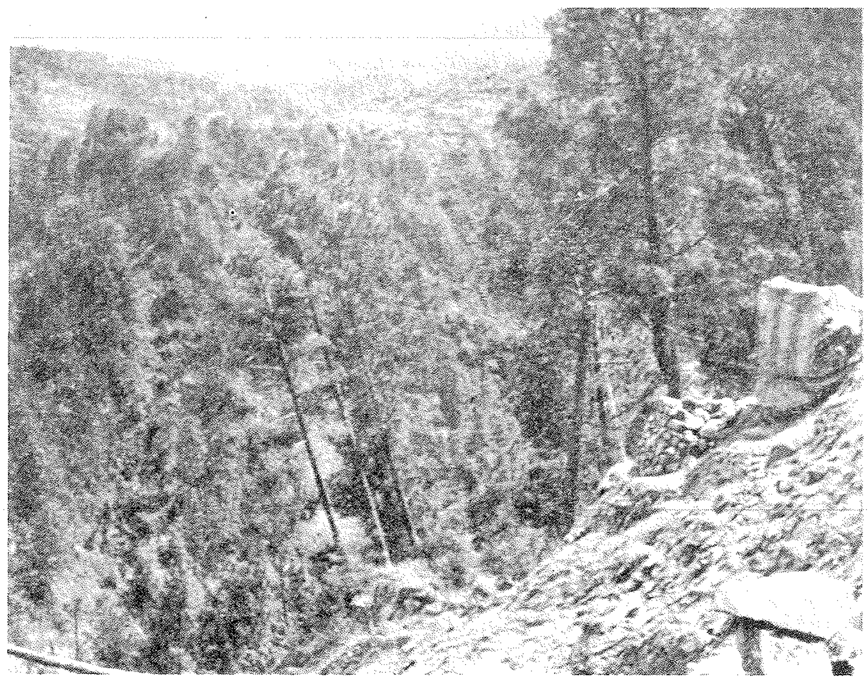

Fig. 29 Tilting of trees indicating surficial creep

situation, the map of the *en-echelon* cracks will delimit the slide area more or less accurately, despite the absence of any physical movement.

Also, cracks can be of help in determining the landslide type with which one is dealing. In a rotational slide, the walls of cracks are somewhat curved in the vertical plane and are concave towards the direction of movement. For the rotational slump block with a sizeable vertical offset, the cracks wedge shut in depth. In the case of a block slide, the cracks are found to be more or less equal in width from top to bottom and there is no narrowing of width in the opening with increasing depth below the surface. This feature is accounted for by the fact that in the case of a block glide, failure begins from the bottom of the block and proceeds upward, towards the surface. The distinguishing feature between a block glide and a lateral spreading, is that lateral spreading is often characterised by a maze of intersecting cracks, whereas block glide shows only a few major cracks in the upper part of the sliding material. Also in a block glide in clay, the cracks are almost vertical regardless of the depth of the sliding plane. On the other hand, in the case of a block glide in rock, the inclination of the cracks would depend on the joint system in the rock.

It is significant to note that by studying the pattern of cracks, a distinction can be made between incipient block glides and rotational slumps. Very often, a pattern of cracks resembling a horse-shoe in place with or without cracks within it, is a sure indication of a rotational failure. On the other hand, if there are cracks parallel to the slope or the rock face, it heralds the development of a block glide.

Yet another feature that is of help to the landslide analyst is the occasional clue given by the general orientation to trees on rotational slumps where, invariably, the trees are bowed uphill, because of the trees tending to grow vertically whereas the stump portion of the trees tend to change or rotate with the rotating land surface as shown in Fig. 28. Fig 29 shows tilting of trees which is indicative of surficial creep.

A description of the surface features of the various parts of active or recently active slides, as an aid toward the identification of different landslide types is given in Table 9 included in Chapter 8.

Chapter 6 TECHNIQUES OF PREVENTION AND CORRECTION

6.1 LIST OF CORRECTIVE TECHNIQUES AND PRINCIPLES INVOLVED

Different types of corrective measures are listed below and details regarding the limits of their application and the principles involved and other remarks are set forth for each of the types listed.

Corrective measure	Application	Principles involved	Remarks
(1) Relocation	All types of landslides	Structure moved to location where mass movement is not present. A permanent solution.	Because of the degree of permanency, a very desirable solution. Very often this is the most economical solution. Its use will depend on obtaining a satisfactory line and grade.
(2) Excavate,drain and backfill	In shallow soil or deep soil in combination with light weight backfill.	The sliding mass is removed and the area stabilized by improving foundation conditions. A permanent solution.	All slide material should be excavated to solid rock, drains placed to intercept seepage, and the area backfilled. Unless highly organic, the same soil can be used for backfill. Good construction methods must be used. Where bedrock is not encountered, stability estimates based on shear values of reworked soil should be made, and a 2 ft. deep granular course should be placed at the bottom.
(3) Drainage-surface, subsurface; tunnels	Subsurface drainage is good for shallow soil. Tunnel can be used for deep soil when water source is deep. Surface drainage should be used for all types preferably.	Removal of water results in (i) lower density of the soil, (ii) higher soil cohesion, (iii) elimination of lubrication of slip plane, (iv) removal of excess hydrostatic pressure, (v) removal of seepage forces. A permanent solution.	Surface drainage should be assured for all types of corrective measures. Generally, drainage should be designed to intercept water before it enters the slide. In an impervious soil, the water source must be reached so as to be effective in controlling movement Subsurface drainage may not be feasible due to the undermining of slide material above the excavation. Subsurface drainage can best be achieved by the use of horizontal drains, Fig. 30.

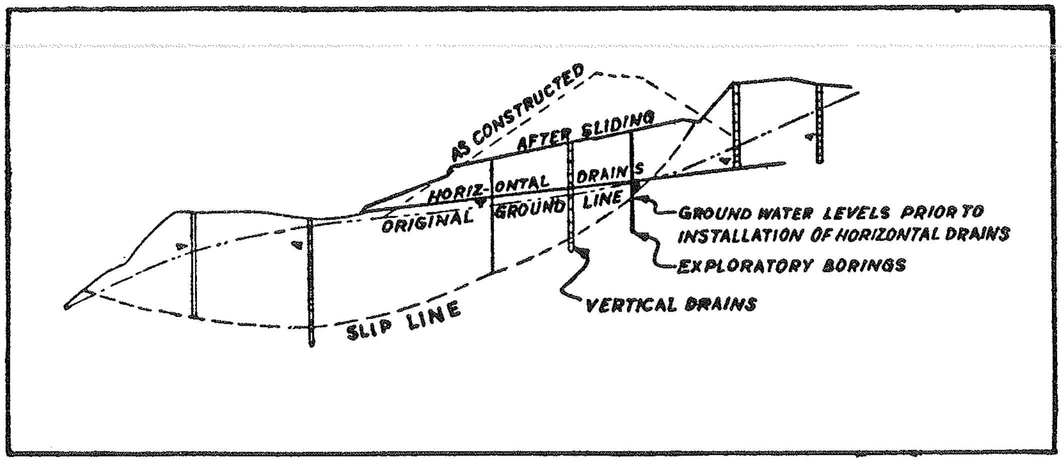

Fig. 30 Slide treatment consisting of horizontal drains and vertical drain wells. (Courtesy of California Division of Highways)

Corrective measure	Application	Principles involved	Remarks
(4) Removal of material.			
Entirely	Is applicable for shallow soil.	Entire slide mass is removed. A permanent solution.	For shallow soil profiles, consideration should be given to a solution involving the entire removal of the slide. For deep soil profiles, the removal of material at the top of the slide will increase stability. For correction by removal, undermining of areas should be considered. The removal of the material at the toe of a slide is rarely desirable, although at times necessary, in order to open a road for traffic. If the only corrective action applied is the removal from the toe, the slide will continue.
Partial at toe	Is applicable to clear road for traffic.	The condition relieved temporarily. An expedient solution.	
Partial at top	Is applicable to deep soil.	A large part of the main source of shearing force is removed. A permanent solution.	
(5) Buttress at toe	When good foundation is available at toe in shallow or deep soil. Buttress should extend below the slip plane.	Large mass blocks the movement and any further movement involves displacement of the buttress. A permanent solution.	An excellent method of correction when the situation at the toe permits-action is that of a retaining device. Much less expensive than cribbing or a retaining wall. The buttress should be founded below the slip surface, for unless the sliding material has an angle of internal friction

Corrective measure	Application	Principles involved	Ramarks
			greater than 15 degrees, the added normal force of a buttress has little influence on the safety factor, Figs. 31 and 32.
(6) Bridging	Steep hillside locations with a narrow failure parallel to road in deep soil.	The troublesome area is bridged and future movement passes under the structure. A permanent solution.	Very satisfactory solution. The foundations must be in bed rock that is not subject to movement or erosion. Too costly for general application.
(7) Cribbing-timber, concrete or metal	Suitable for shallow soil foundations.	A retaining mass, with or without additional lateral restraint, placed in path of the mass movement - furthur movement involves displacement of the retaining mass. A permanent solution.	Good method where applicable but relatively costly. Less stable foundation required than for retaining wall as shifting is permissible before cribbing. In slide areas, the resistance required can be estimated by using the formula for the safety factor. Not recommended in creep and flow areas.
(8) Retaining wall of stone or concrete	Shallow soil where good foundations are available.	A retaining mass with lateral restraint, placed in path of the mass movement — further movement involves displacement of the retaining wall. A permanent solution.	The applications are similar to to those of cribbing. The cost is a deterrent. Walls are advantageous in urban area. A wall requires a foundation in bed rock or good soil below the slip surface. Standard practice is to include weep holes in designing the wall. The formula for the safety factor may be used to estimate resistance required to lateral thrust. Retaining walls in creep and flow movements may take full weight of soil.
(9) Piling.			
Floating	Used in shallow soil to hold the slide mass temporarily.	Piling offers a retaining influence. For soil masses, the piling may tend to 'pin' the moving and stable materials and bring about a more lengthy slip-	Floating piles do not appreciably penetrate bedrock or the slip-surface. Such methods have lasted from one to 10 years. With a slip surface at bedrock, the fixed piling penetration for a distance of 3 to 5 ft. into the bedrock offers resistance
Fixed with no provision for extrusion	Suitable in shallow soil with nume-		

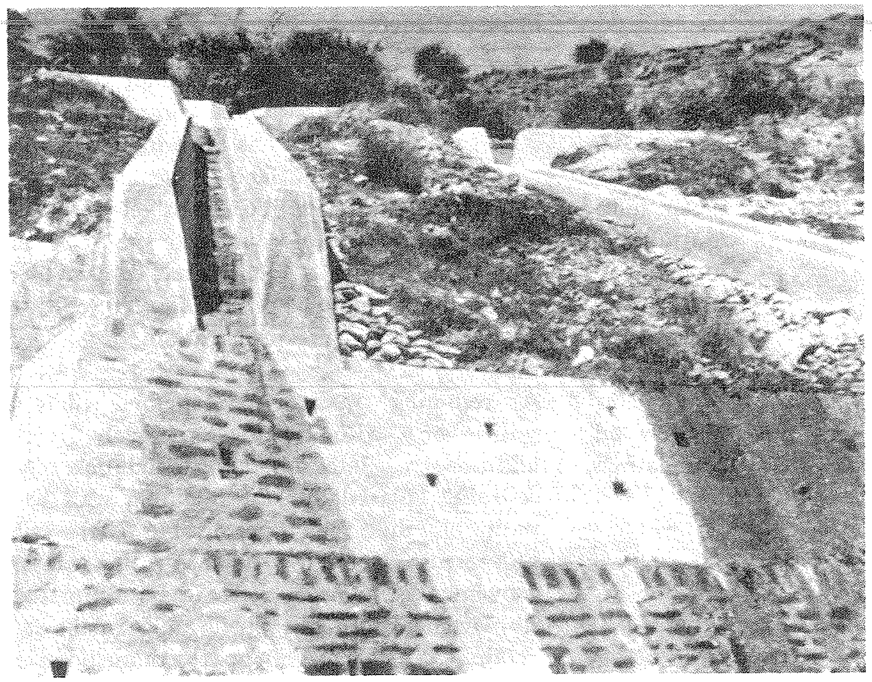

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Fig. 31 Drop channel with buttress wall

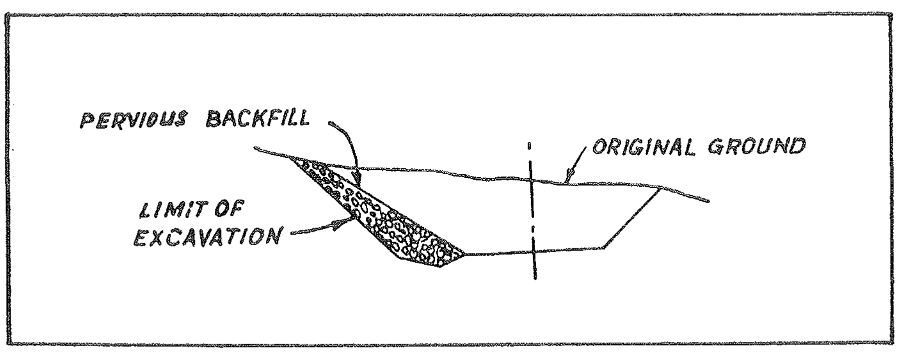

Fig. 32 Modified buttress. Pervious blanket of gravel or rock prevents sliding of wet cut slope by providing drainage of slope and also by increasing strength of slope by buttress action. (Courtesy of California Division of Highways)

Corrective measure	Application	Principles involved	Remarks
Fixed with provision for preventing extrusion	rous large rocks. Suitable in shallow soil.	surface with a corresponding increase in shearing resistance. Floating piles are expedient. All others are permanent solutions.	equal to the force required to bend the piling. If the soil is wet and fine grained, movement in the area is likely to be due to extrusion between the piles, for piling in deep soil where the base of the pile penetrates 5 ft. past the slip surface, the recommendation is to estimate the forces on slip surface curving under the piling. Stability can be estimated by obtaining safety factor. For piling with provision for preventing extrusion (such as I-beam and cribbing combinations) the principles of retaining wall apply.
(10) Sealing joint planes and open seams	Deep source of water.	The same benefits are sought as for drainage solution. A permanent solution.	It is essential to locate the source of water and to block its entrance by grouting. Difficult to achieve and to estimate quantities. The effect of blocking the water should be studied for the area above and around the slide. This method of correction is still experimental.
(11) Cementation of loose material	Permeable material only.	An attempt to improve the shear characteristics of the soil.	Stability is achieved by cementing the slide material. In order to produce deep soil stabilization, a permeable material is essential.
(12) Chemical treatment-flocculation	Permeable material.	Same principles as above, except that chemicals are used. A permanent solution.	This is essentially the same as cementation, except that the chemicals are used.
(13) Tie-rodding slopes	Shallow soil with large rock fragments.	The retaining device insufficient to withstand movement is anchored to bedrock. A permanent solution.	
(14) Blasting	Shallow soil underlain by good rock.	An attempt is made to disrupt the sliding surface as well as to develop a drainage	

Corrective measure	Application	Principles involved	Remarks
		system. An expedient solution.	
(15) Reshaping slide material	Used where face to slide has open fissures. Benching likely to cause ponding of water.	See item No. 3. Method generally used in combination with other actions.	Area improved by sealing surface so as to prevent the entrance of water.
(16) Slope treatment	Shallow soil in combination with other methods or any erosion problems.	See item No. 3. Method generally used in combination with other actions.	Area improved by sealing surface so as to prevent the entrance of water and to minimise erosion. Use seeding or sodding, blanketing with cinders, guniting or sealing of surface. Use asphalt mulch technique.
(17) Light weight fill	In cases where loading is critical.	Reducing of the weight causing movement. Rarely a solution in itself.	The heavier soil is replaced with a lighter permeable material. Often helpful when it is desirable to hold a given grade.

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NOTE: There are a few other miscellaneous methods available but most of them are yet to pass into routine use in the field.

6.2 REMEDIAL MEASURES GENERALLY RECOMMENDED FOR THE LANDSLIDE TYPES DISCUSSED IN CHAPTER 4

Detritus Slides

The most effective means of preventing the danger of detritus slides along gentle slopes is adequate drainage. Since the layer of detritus is commonly shallow (not more than 20 ft. deep) detritus flows can sometimes be stopped by driving piles through the moving material into the firm base. Normally, several rows are driven at right angles to the direction of movement.

Since no slides in detritus can take place without an abundance of water, the possibility of such slides can be eliminated by preventing temporary saturation. This is best done by means of a catch water drain running along the upper boundary of the area to be protected and covering the surface of the slope by means of an impermeable material, or by providing a vegetative cover.

Slides in Sand

Since flow slides in sand occur only when the sand is very loose, the tendency towards sliding can be mitigated by densifying the sand. This can be accomplished by different means such as by driving sand piles or by exploding small charges of blasting powder in the interior of the loose sand mass.

Slides in Loess

Slides in this type of deposit, it is believed, can be corrected by preserving the strength of the loess by means of suitable bituminous treatment preventing the ingress of water into the soil mass.

Slides in Stiff Clay

Since stiff clays are known to suffer a decrease of strength over a period of many many years, from a high initial value at the time of the cut to a low value at the time of the slide, it would seem uneconomical to select the slope angle of a cut in such soils on the basis of the ultimate value of the shearing resistance. However, it is desirable to delay the deterioration of the clay as much as possible by draining the strip of land adjoining the upper edge of the cut for a width equal to the depth of the cut and to treat ground surface of the cut area to reduce the permeability. If local slides occur, they are to be subsequently corrected by local repairs. If delayed slides would endanger life or cause excessive damage to property, the slopes should be provided with reference points to enable periodic level observations to be made, in order to detect possible movements in advance of failure so that the slopes in the danger sections may be flattened.

In similar circumstances, there have been instances where stone rubble drains have been successfully used to prevent movements at danger sections. These drains are built up of dry stone masonry or rubble. These drains run up and down the slope at a spacing of about 20 ft. The drains are constructed in trenches dug out to a depth equal to that to which the clay has been softened. A concrete foot-wall supports the lower ends of all the ribs. The beneficial effect of this type of construction is often attributed to the function of the ribs as drains, but it seems more plausible that the principal function of the ribs is to transfer part of the weight of the unstable mass of clay through side friction to the footwall.

Slides in Clays, Slopes Underlain by Layers or Seams of Sand or Silt

The solution in such circumstances would be to prevent the building up of excess hydrostatic pressure in the sand or silt seam or layer. This can be accomplished in different ways, such as by the use of relief wells, vertical sand drains or horizontal drains as the case may be.

6.3 DESIGN CONSIDERATIONS AND DISCUSSION ON CHOICE OF CORRECTIVE AND PREVENTIVE TECHNIQUES

Avoidance of Potential Landslides by Relocation, Bridging etc.

It is not often practicable to avoid a potential landslide by changing the location of a proposed highway but the possibility should not be overlooked. In some cases it may be shifted into stable ground by a slight change in alignment.

Where the proposed excavation will cross formations that are susceptible to bedding plane slides, the slide hazard can sometimes be reduced by adjusting the alignment so that the cut slopes intercept the beds at a more favourable angle to the bedding planes. In some places, for example, it may be possible to choose the opposite side of a valley or hill where the bedding planes of the rock will dip away from the cut slope rather than dip towards the cut, Fig. 33.

While locating new lines of transportation, it is worth keeping in mind some of the typical situations conducive to landsliding, induced by proposed cuts or fills. These are:

(1) Restriction of ground water flow by sidehill fill.
(2) Overloading of relatively weak underlying soil layer by fill.
(3) Overloading of sloping bedding planes by heavy sidehill fill.
(4) Oversteepening of cuts in unstable rock or fill.
(5) Removal of cuts of thick mantle or pervious soil if such pervious soil happens to be a natural restraining blanket over a softer core.
(6) Increase in seepage pressure caused by cut or fill that changes direction or character of ground water flow.
(7) Exposure by cut of stiff fissured clay that is liable to soften and swell when exposed to surface water.
(8) Removal of mantle of wet soil by sidehill cut. Such a cut may remove toe support causing soil above cut to slide along its contact with stable bed rock.
(9) Increase in hydrostatic pressure below surface of a cut in silt or permeable clay.

All these factors should be carefully considered when evaluating the probable effects of new construction.

Recognition and consideration of these factors are essential in determining whether or not an attempt should be made to forestall a potential landslide by avoidance.

Prevention of landslides by avoidance does not always require a change in alignment or location of a road. In some situations, a mere revision in the grade line of the proposed highway may prove effective in preventing slide movement. For instance, where the most desirable grade line for a new highway construction requires undercutting of an unstable slope, it may be possible to adjust the profile grade of the road so as to avoid any excavation at the toe of the hill and instead, to provide additional support by construction of an embankment which will act as toe support or 'strut'.

If there is no way to avoid a potential landslide and if preventive treatment will not assure stability, it will sometimes be necessary to construct a bridge across the unstable area. Since the cost of a bridge is usually prohibitive, extreme care must be exercised in designing a structure that will not itself be damaged by moderate slide movement.

Thus, due to the degree of permanency involved, relocation should be regarded as a desirable solution. Very often it is the most economical solution although its use will depend on obtaining a satisfactory line and grade.

6.4 EXCAVATION METHOD

Excavation methods invariably contribute to increased stability of the soil mass beneath a slope. The principal methods, Fig. 34, used for prevention or correction are:

(1) Removal of head of slide.
(2) Lowering of the grade line.
(3) Flattening of slopes including benching of slopes.
(4) Complete removal of all unstable material.

Removal of Head of a Slide

Principle

The method aims at unloading or taking away a relatively large quantity of material from the head of a landslide, thereby reducing the activating force.

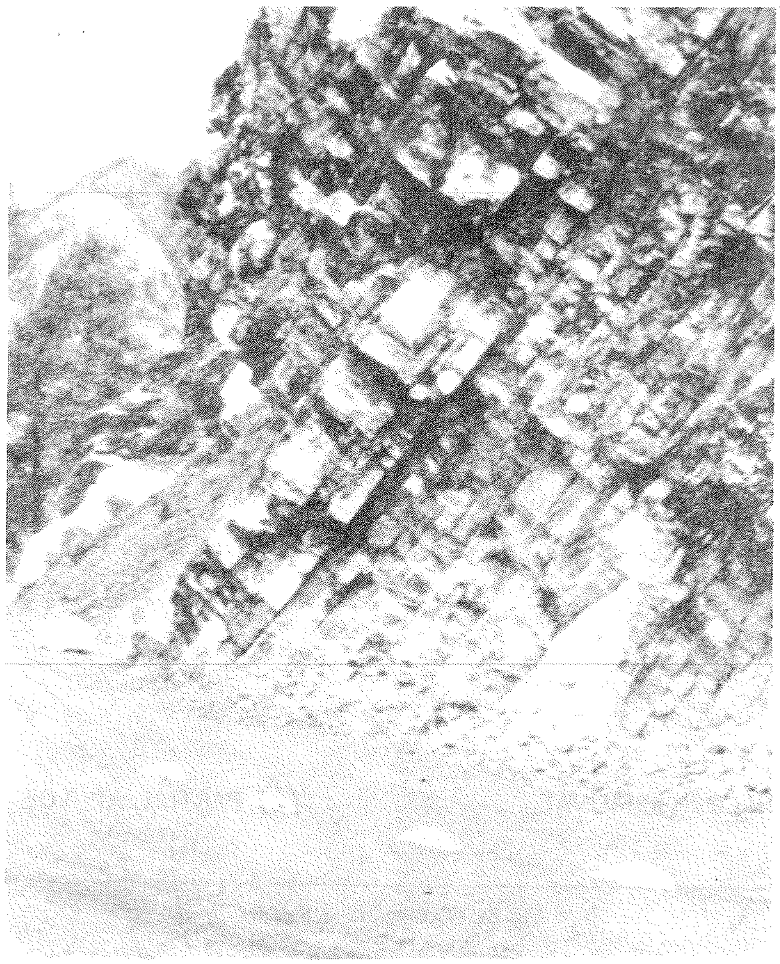

Fig. 33 Dip favourable for the road and unfavourable for the same road beyond the turning

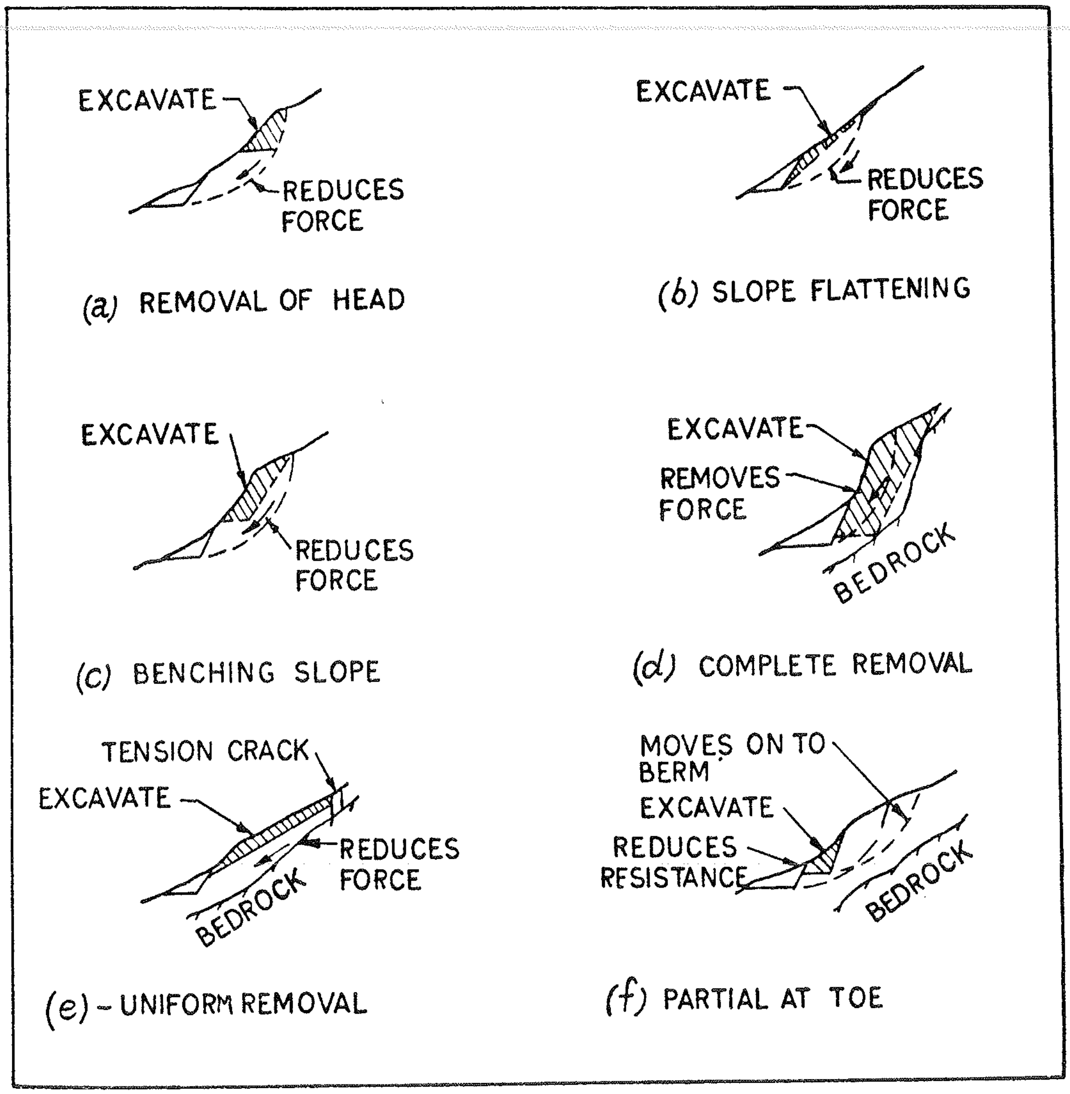

Fig. 34 Excavation techniques

Prevention of landslides

This method is seldom applicable to prevent landslides although it has been occasionally used in controlling potential slides in talus material. The method is essentially applicable to treatment of an existing slide.

Correction of landslides

The method is suitable.

Situations where the method will prove promising

(1) The method is a very practicable one provided the quantity to be removed is not excessive.

(2) The method is best adapted for distinctly rotational slides. In slides with curved slip planes, the gravitational force in the upper part of the slide is particularly large and therefore unloading of the material is apt to give tangible corrective benefits.

Situations where the method will be of questionable benefit

(1) The method is inapplicable for flows.
(2) The method is not suited for correcting planar slides with straight surfaces of rupture.

Design considerations

(1) Methods of stability analysis involving application of soil mechanics principles can be advantageously used to evaluate the benefits of removal and also the quantities to be removed.
(2) The quantity to be removed from the head generally works out to be 1 to ~~20~~ 2 times the quantity removed or to be removed from the toe of the landslide. This will give a relatively flat surface (15 : 1) at the head.
(3) In situations where there is little or no indication that removal of toe material has been or will be accomplished by man or nature, approximately 0.15 to 0.25 times the moving mass represents the quantity to be removed from the head.
(4) Removal of head should not in its turn cause movements above the excavation.

Flattening of Slopes including Benching of Slopes

Preventive measures in excavation areas consist primarily of proper slope design and drainage.

It is generally more economical to design the excavation with slopes that will minimise sliding rather than to excavate steep slopes and subsequently to flatten them after sliding has occurred.

In using this method as a preventive measure, a study of existing cut slopes of similar material in the locality is helpful but the comparison should take into account whether the heights and the materials involved in the existing and in the proposed cuts are at all comparable. For example, 1 : 1 slope in a given formation may be stable for a 30 ft. high cut but the same slope might prove too steep for the same type of soil in a cut 100 ft. high.

In homogeneous soils, the strength can be reliably determined by laboratory testing and slopes can be designed on the basis of principles of soil mechanics. However, this is not a reliable method in non-homogeneous soils and rock formations where the strength cannot be determined with any degree of accuracy. Even in such cases, a knowledge of the character of the material, subsurface water conditions and a rough stability analysis can combine to help decide upon the proper slope angle. This can be supplemented by a study of existing cut slopes.

It should be recognised that flattening of rockslopes may not be a lasting benefit because the geologic structure of the rock causes it to assume steeper slopes with the flux of time.

'Excavation' is economical and practicable as a preventive measure, whereas it costs relatively much more in the correction of landslides. This is because, unit costs for the relatively large quantities of earthwork involved are generally lower on new construction projects than on repair jobs.

For purposes of correction, excavation techniques are frequently used in all classes of landslides. They are best suited to slides moving downslope towards a road and not for slides that undermine a road on its downward slope. Before a particular method of excavation is decided upon, one should know whether the failure is to be classified as a fall, a slide or a flow and whether the slip-surface is curved or straight. It is also essential to know if the failure developed at the toe of an excavation and thereafter proceeded upslope or the movement developed simultaneously in the whole slide area. The need for such information will be better illustrated in the succeeding chapters.

There are three types of slope designs possible, namely, the uniform slope, the variable slope and the benched slope.

Uniform Slope and Slope with Varying Angles

A uniform slope is adopted from ditch-line to the top of the slope. For most small cuts less than 20ft. high, uniform slopes are probably the cheapest solution. For large cuts, the geology of the region should be taken into account in deciding on the angle of a uniform slope. If different kinds of rocks are interlayered, a uniform slope may result in improper design for one or more of the layers.

Slope angle is related to the height of the cut and the geologic structure of the material.

In every locality, one should observe the maximum height at which the weaker rock materials tend to maintain stability on a given slope. This observation will be very helpful in determining the proper slope angle.

If the slope consists of straight sections with varying slope angles, it permits the use of proper and most economical slopes for each of the geologically varying sections besides reducing erosion along long slopes. This method however will entail a detailed geological investigation and occasionally it may prove to be economical to over design or to adopt a uniform slope based on less detailed investigations.

In soil slopes, such methods including benching are rarely suited to flows or to slides with straight slip-surfaces.

The method is usable on embankments and in cuts which have caused small slides above the top of the cut due to undercutting of the slope. Large slides are better tackled by the 'removal of head' method. Most talus soils are likely to be stable on 2:1 slopes for a cut upto 20 ft. in height but may require 3:1 slopes for cuts greater than 30 ft. in height.

Benching of Slopes

This method involves straight slopes separated by near-horizontal benches.

Generally, slopes constructed with benches or 'berms' are considered preferable to equivalent uniform straight slopes.

Benching produces increased stability by dividing the long slope into segments of smaller slopes connected by benches. The proper width of bench can be estimated by analysis of stability of slopes for a given soil. It is generally observed that the width of benches should be equal to atleast 25 ft. to enable the slope segments to act independently.

The choice of a benched slope assumes the inevitability of a certain amount of disintegration in newly exposed rock faces. The slope angle between the benches can either be uniform or variable. The width of benches, the vertical height between benches and the slope angle should be determined. The main advantages of this method are:

(a) If the geologic characteristics of the bed rock cannot be determined readily, this method comes in handy compared to the other excavation methods.

(b) In shales and in similar rocks, susceptibility to erosion is cut down since benches reduce the velocity of water running down the slope.

(c) Construction is easier since steeper slopes are feasible with benches.

(d) The weathering products are intercepted by the benches and the benches serve as clean-off areas. The debris falling on the benches can be removed periodically making room for additional weathered material. But it may be preferable in some cases to let the debris remain as insulation against continued weathering of bed rock and ultimately to seed the surface.

(e) For most materials, the slopes between the benches can be steeper than the ultimate.

Principles of Design

Benches can either slope away from the roadway or slope toward the road.

A roadward slope permits immediate run off of surface water. Consequent upon improved drainage, the tendency to sliding in clayey debris accumulating on the roadward sloping benches is reduced. On the other hand, in a bench sloping towards the road, the run-off can induce serious erosion of the slopes lying below it.

In the case of a bench sloping away from the road, the accumulating clayey debris on the bench can hold water, remain plastic and may eventually slide. But where rock is involved, a bench that slopes into the hill will resist sliding of rock debris whereas a roadward slope may include movement of debris on to the underlying slopes.

As a guiding rule, it can be stated therefore, that the bench should slope away from the road where there is no clayey material involved, preferably with a provision for longitudinal drains along the inner edge of the bench. Whenever clayey soil or debris is involved, the slope should be towards the road. 165

A typical multiple-benching in bed rock is shown in Fig. 35. Likely values of height and width of benches and slope angles for different types of rock is set out in Table 7 to serve as a guide.

TABLE 7. **LIKELY VALUES OF HEIGHT AND WIDTH OF BENCHES AND SLOPE ANGLES FOR DIFFERENT TYPES OF ROCKS (REFER TO FIG.35)**

	Height between benches, ft.		Width of benches, ft.		Backslopes (horizontal : vertical)	
Type of rock	H_a	H_b, H_c, etc.	W_a	W_b, W_c, etc.	S_a	S_b, S_c, etc.
(1) Major cut in shale with interbedded sandstone.	5-20*	20-30	0-30	20-35	$\frac{1}{2}$:1	$\frac{1}{4}$:1 to $\frac{1}{2}$:1
(2) Major sandstone cut.	10-30*	30-40	0-20	20-30	$\frac{1}{3}$:1	$\frac{1}{4}$:1
(3) Major cut in sandstone underlain by shale.	10-30*	20-40	0-25	20-35	$\frac{1}{4}$:1	$\frac{1}{4}$:1 to $\frac{1}{2}$:1
(4) Moderate cuts in sandstone and shale	10-40*	20-40	0-20	20-30	$\frac{1}{2}$:1	$\frac{1}{4}$:1
(5) Major cuts in shale	10-25*	20-30	0-30	20-30	1:1	$\frac{1}{3}$ 1 to 2:1

*Use minimum if $W_a = O$.
Source: H.R.B. Special Report 29.

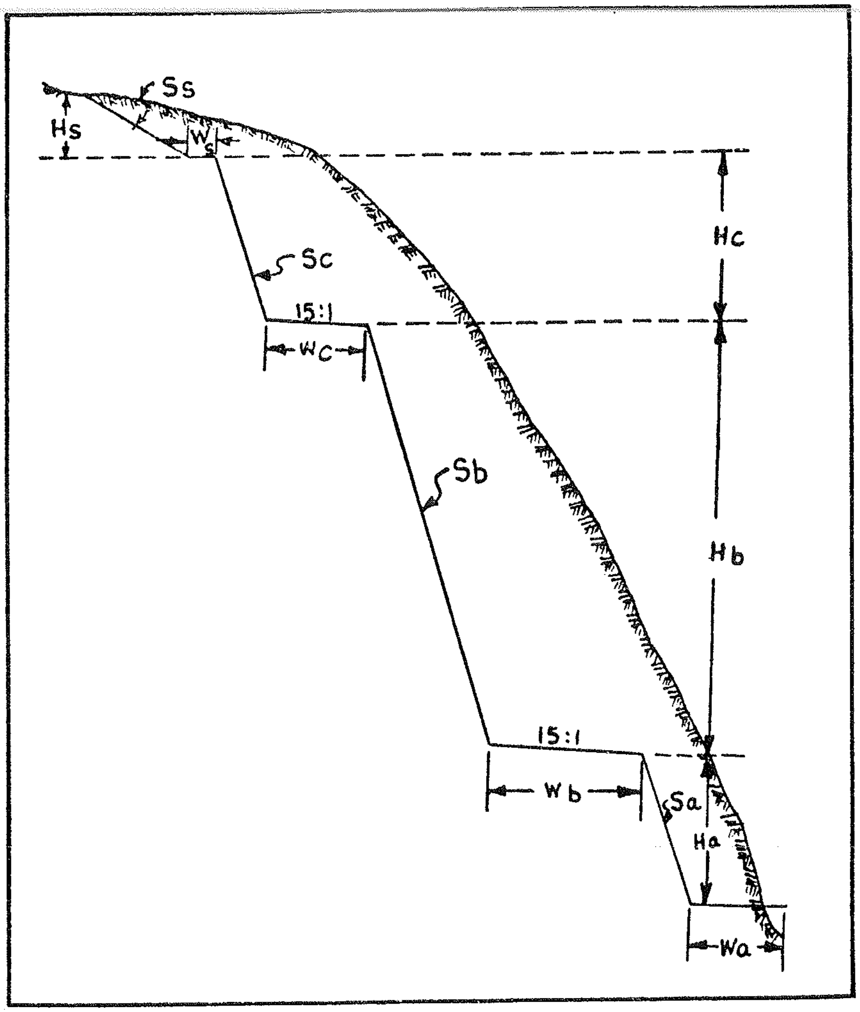

Fig. 35 Criteria for bed rock slope design as used by West Virginia State Road Commission. This technique employs a combination of benches and relative steep back slopes (Refer to Table 7)

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The geologic structure, such as the attitude of bedding planes or joints has a very important effect on slope stability. In a given case, the quantity of weathered products to be expected on the benches is the deciding criterion for fixing various dimensions. Local experience and observation are essential guides to design.

The benches should be constructed with a V or gutter section with a longitudinal drainage grade and with suitable catch basins to carry the water down the slopes. Occasionally, paving of the ditch may be necessary to reduce erosion or to prevent percolation of water into pervious areas on the benches.

In the case of soils which are fairy homogeneous, the width of benches can be evaluated on the basis of the principles of soil mechanics.

6.5 DRAINAGE METHODS

Surface Drainage

Prevention

All precautions should be taken to prevent the surface runoff water from entering a potentially unstable area.

Whenever a new construction crosses an old landslide, its surface should be reshaped as necessary, to provide good surface drainage. It has been commonly observed that the culverts get clogged up due to poor maintenance as shown in Fig. 36.

Excessive or unnecessary removal of vegetation will promote surface erosion.

Sealing of cracks on the surface of the slope in any type of landslide will prove beneficial since it will prevent the ingress of surface water into the slide mass and mitigate frost action.

For potential landslides, surface drainage measures represent a good investment compared to any other type of preventive treatment although surface drainage may have to be used invariably in conjunction with other types of treatment.

Methods of surface drainage include (a) reshaping of slopes, (b) construction of paved ditches, (c) installation of drain pipes, (d) paving or bituminous treatment of slopes.

Correction

Surface drainage will prove quite useful and can be used at little extra cost in conjunction with other corrective methods.

Use of surface drains along the outer periphery of the moving area is particularly desirable. The surface drains help to intercept the runoff from higher ground. 167

If such surface drains are likely to be clogged by debris from above, a pipe should be placed in the ditch to ensure that the water will not be trapped. The ditch should be sloped so as to drain off the water quickly or else the base of the ditch should be sealed with an impermeable material. If this is not done, the ditch may prove detrimental inasmuch as it will serve to feed water into the slide area.

Seeding or sodding or vegetative turfing by means of the asphalt mulch technique can be used. Surface drainage techniques such as these are not adequate by themselves for providing correction except in conjunction with other corrective procedures.

Sealing of cracks can be done by regrading the surface. Occasionally, individual cracks may be sealed more economically and rapidly by hand-filling with clay, bituminous materials etc. Immediate attention to crack-sealing seldom goes profitless in correcting a slide although additional corrective measures will be desirable in many instances.

Subdrainage

The drainage of subsurface water tends to produce a more stable slope for the following reasons

(1) Seepage forces are reduced.

(2) Shear strength of the soil is increased.

(3) The excess hydrostatic pressure within the soil mass is reduced.

(4) The unit weight of the soil is reduced which under particular situations may result in reducing the slide inducing forces.

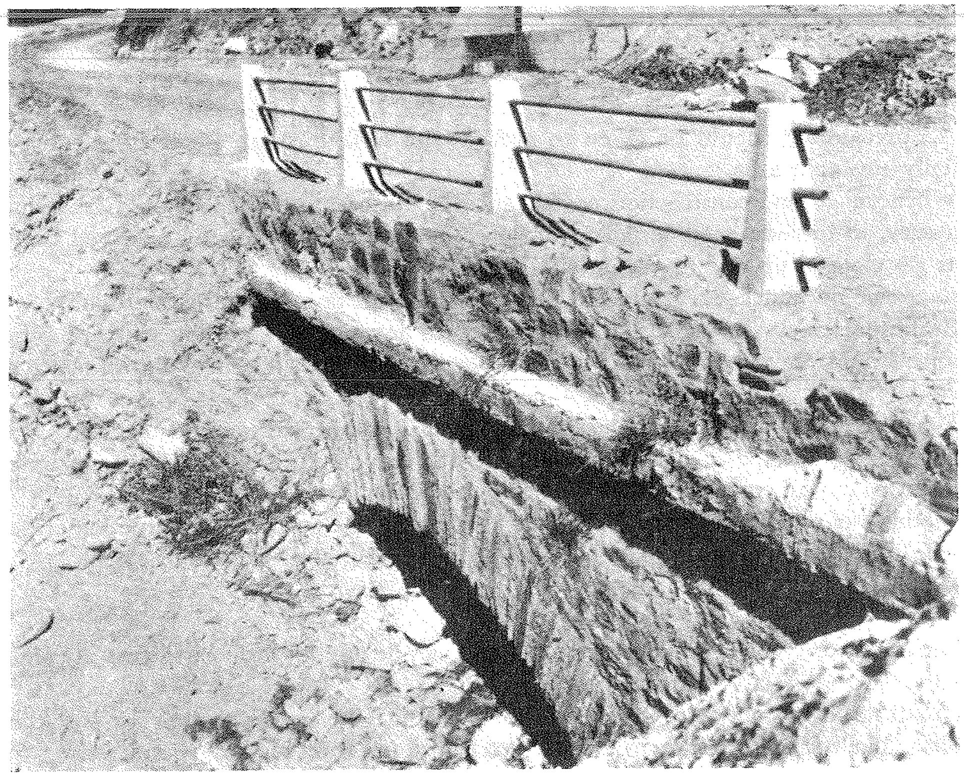


Fig. 36 Culverts clogged up due to poor maintenance practice

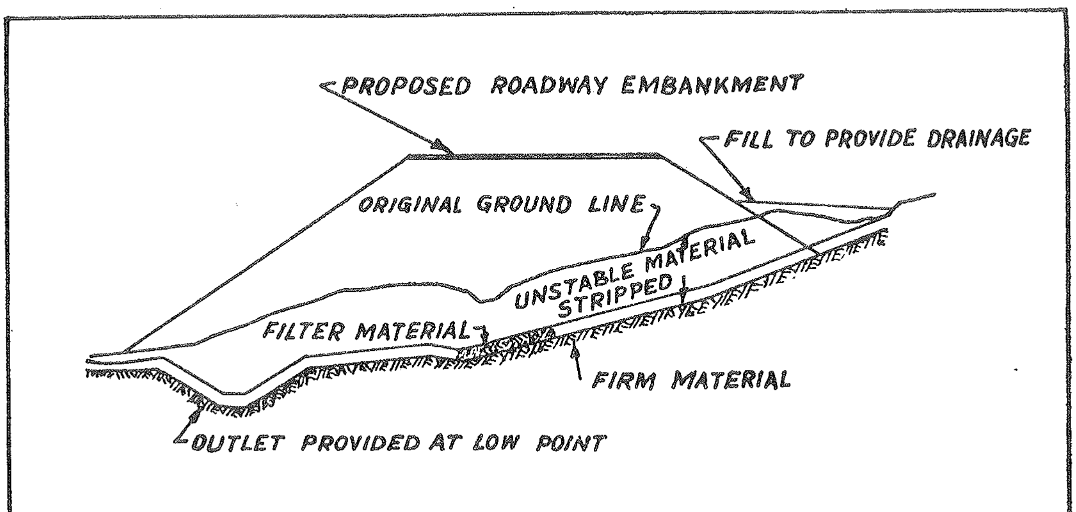


Fig. 37 Stripping as a slide prevention measure—a typical cross-section of highway showing stripping of unstable material before constructing embankment

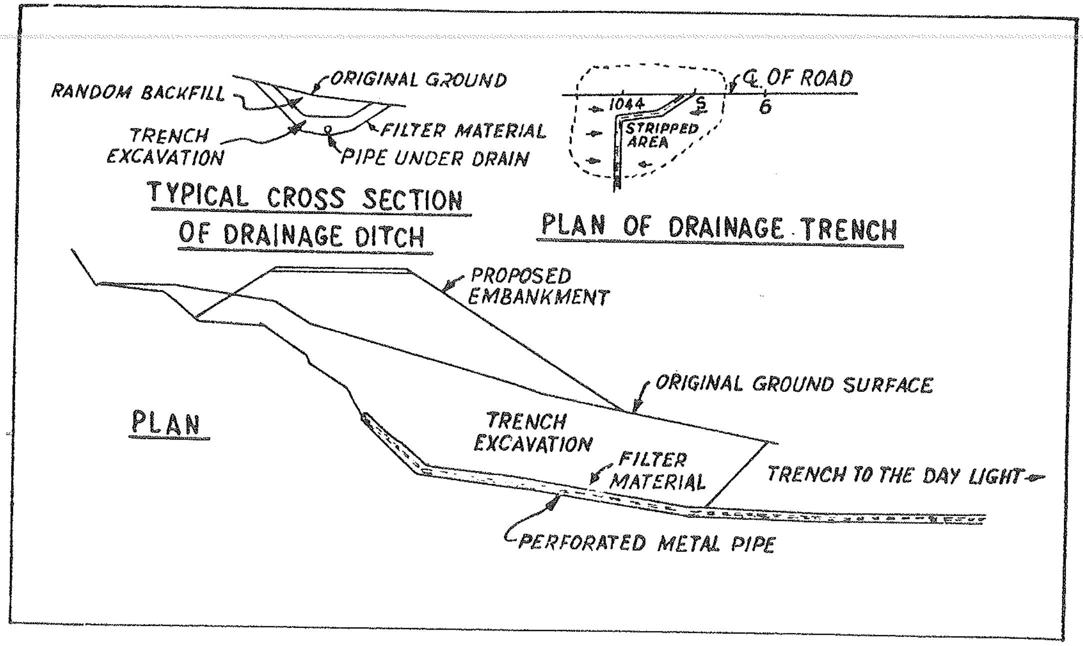


Fig. 38 Slide prevention by combination of stripping and drainage trench. Plan and cross-section of preventive treatment consisting of stripping unsuitable soil and constructing drainage trenches. (Source —Sketch furnished by C.P. Sweet: courtesy of California Division of Highways)

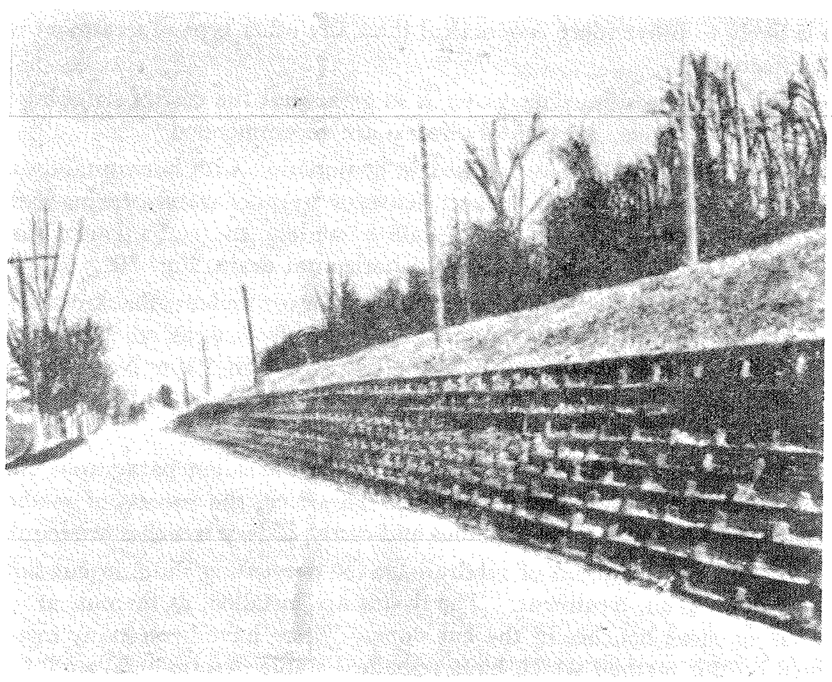


Fig. 39 Timber cribbing for slide control

Prevention in Embankment Areas by Subdrainage

Two factors require consideration in the investigation of possible slipouts (a) weak zones in the foundation soil which are likely to be overloaded by the proposed embankment load, and (b) subsurface water which is liable to result in the development of hydrostatic pressure or reduce the shear strength of the soil to the extent of initiating a slide.

Methods of Preventing Highway Slipouts

(1) If a surface layer of weak soil is relatively shallow and is underlain by stiff or dense soil, the unstable material may be stripped, as illustrated by Fig. 37. If there is possibility for seepage to develop during wet seasons, a layer of pervious material may be placed before the embankment is constructed. Clean gravel, free-draining sand or other suitable local materials may be used. It may be necessary to use a drain pipe if springs or concentrated flows are encountered.

(2) If stripping the unstable material proves to be uneconomical, deep drainage or stabilization trenches are sometimes excavated with power equipment with the steepest side slopes that will remain stable during the construction period. The trenches should extend below any water bearing strata and into the firm base. After placing a layer of pervious backfill material on the bottom and side slopes, together with an underdrain pipe at the bottom, the trench is backfilled and the embankment constructed, Fig. 38. If the area to be treated is small, one trench normal to the centre-line of the road may suffice. If the area is large, a herringbone pattern of drains would prove suitable. Besides providing subdrainage, the trenches add to the structural strength of the foundation.

Although the cost of this method increases rapidly with depth, this method of slipout prevention is likely to prove more economical than any other type of treatment which might be equally effective.

(3) If the depth to subsurface water is so great that the cost of stripping or drainage trenches proves prohibitive, horizontal drains are recommended.

(4) Vertical drain wells may be used in conjunction with horizontal drains, if there is need to provide a drainage path between lenses or strata of water-bearing material which are separated by impervious strata. If installed under an embankment an outlet for vertical drain can be provided by means of a horizontal drain, Fig. 30.

(5) Vertical drain wells are also recommended for use under embankments for purposes of accelerating consolidation, through drainage of water from weak compressible foundation soil. For design details, please refer to CRRI Road Research Paper No. 54.

Prevention by Subdrainage in Cuts

All of the subdrainage methods discussed in the preceding paragraphs could as well be applied to prevention of slides in cuttings. However, the success of methods such as drainage trenches etc., can be rather dubious and costly, if deep trenches are required.

The most successful method of subdrainage for preventing slides in cut slopes is likely to be the horizontal drain treatment. The drains are installed, as the cuts are excavated, often from one or more benches in the cut slope. There have been many instances of cut slopes drained by this method which have remained stable despite unfavourable soil formations and the presence of large amounts of subsurface water.

Correction by Subdrainage

(1) Horizontal drains provide the most promising type of subdrainage for correcting a slide. For details of the technique, please refer to Article 6.8.

(2) Drainage trenches or intercepter drains can be used for the same purpose as horizontal drains. Trench type drains are generally limited by practical consideration to those places where water can be intercepted at depths less than 10 to 15 ft. It is most important that the drain pipe be based on unyielding material which generally means that it must be below the slip plane, lest subsequent movement should tend to break or bend the pipe and disrupt drainage before the benefits of drainage could be availed of.

(3) Vertical sand drains are generally to be recommended for use in conjunction with horizontal drains so that lenses of permeable material are connected vertically by the sand drain to remove the water. This approach is specially best suited to correcting landslides that contain lenses of permeable sand within less permeable material.

6.6 COMPLETE REMOVAL OF ALL UNSTABLE MATERIAL

As a preventive measure, the removal of all unstable material is usually not necessary and is seldom economical except for very small masses.

As a corrective measure, this method is usable in all types of slides but its practical limitation will be based on the size of the moving mass. For most slides, this technique may prove prohibitive in cost.

The method is best adapted to situations where the road to be protected is at the toe of the slide. If the road is located in any other part relative to the landslide, this method will be of no avail.

6.7 RESTRAINING STRUCTURES (RETAINING WALLS ETC.)

Restraining structures like retaining walls, cribs, sausage walls, rock buttresses are very often used to correct a landslide. It is found that they have been used with varying degrees of effectiveness, ranging from partial success to total failure in many cases. Those situations in which restraining structures are likely to be both practicable and economical, and particular situations in which such structures are likely to be unsuitable, are set out below:

(1) Retaining devices are seldom applicable for correction of falls and flows.
(2) Rock or earth buttresses, cribs and retaining walls, can be used to correct small slides especially rotational ones, but are not, generally speaking, effective on large slides. Fig. 39 shows timber cribbing for slide control.
(3) Retaining walls and cribs are, however, always useful for purposes of underpinning foundations of structure, in any type of slide.

The indiscriminate use of retaining walls or their misuse is attributable to the failure on the part of the investigator to distinguish between slides and flows. In the case of slides, the retaining wall provides a significant amount of shearing resistance to sliding, besides contributing to improved drainage near to toe of the slide mass.

Retaining devices placed in the path of a flow slide receive the entire force of the moving mass because of the fact that there is little inherent resistance of the soil involved in the flow. Fig. 40 shows the use of retaining wall to protect a highway.

Fig. 41 illustrates a method for reducing damage from rock fall by protection with steel wire mesh.

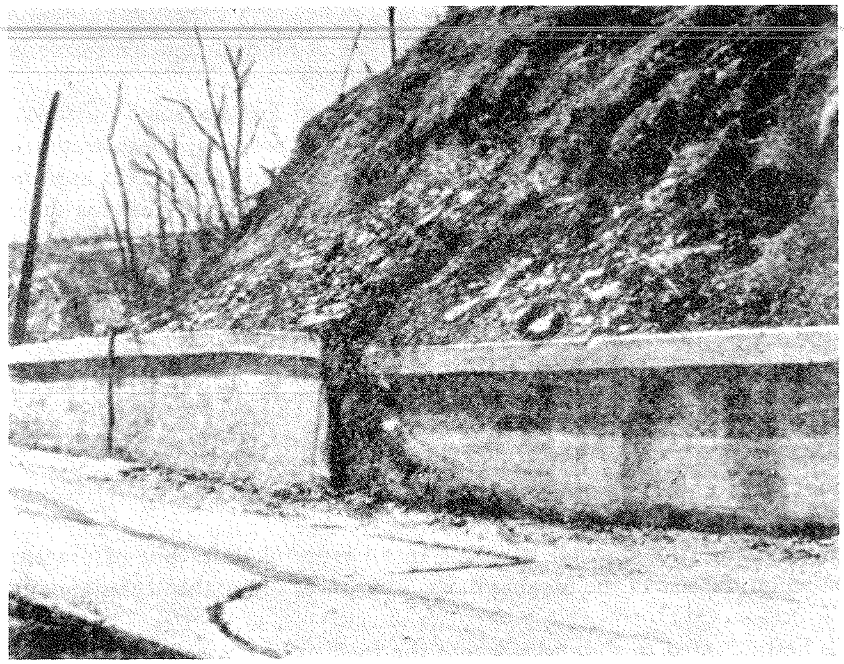
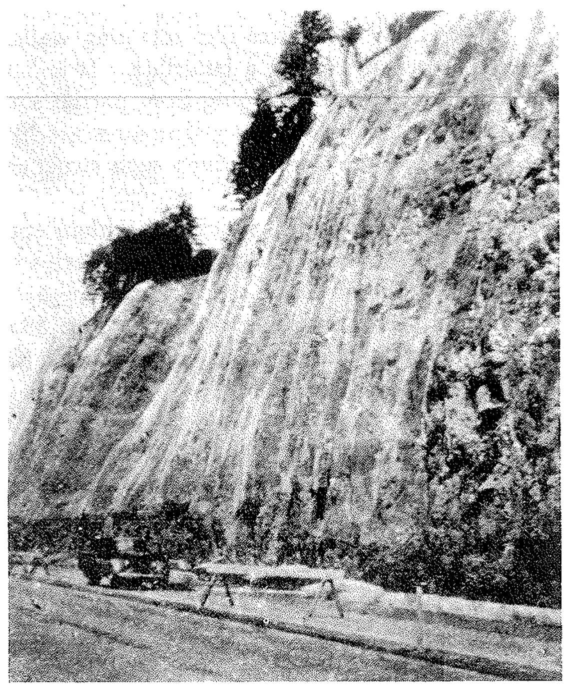


Fig. 40 Retaining wall as a protection of the road

Fig. 41 Wire mesh covering the slope to protect the highway from damage against rockfall (Courtesy of American Hoist and Derrick Company, St. Paul, Minn.)



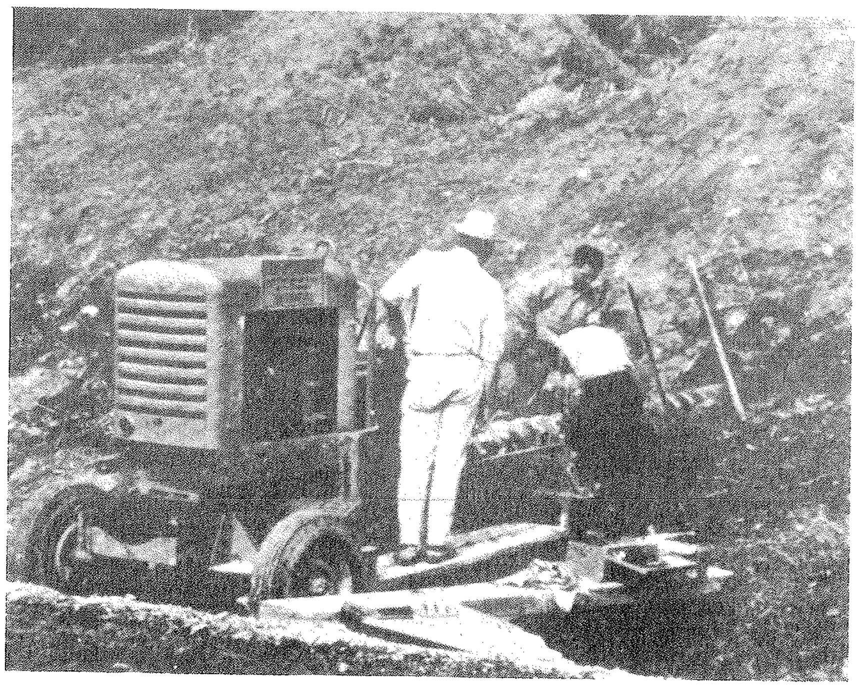


Fig. 42 Installation of horizontal drains

6.8 HORIZONTAL DRAINS

The use of horizontal drains represents a relatively new approach in the control of landslides.

Basically, horizontal drains are holes or borings that are drilled into an embankment or a cut face. They are made usually on a slight plus grade and cased with perforated or porous liners.

Experience has shown that the horizontal drains are effective under a wide variety of soils and geologic, topographic, climatic and ground water conditions. They are to be invariably used in combination with other measures to prevent or correct slides and slipouts.

Purpose

The principal function of horizontal drains is to remove excess subsurface water from hillsides, cut slopes and fills. They provide channels for drainage of subsurface water either from the mass of sliding soil or from its source in the adjacent area.

Benefits

The removal of subsurface water tends to produce a more stable condition in several ways:

(a) Seepage forces are reduced. These forces are not necessarily in the direction of sliding, but by and large, they happen to be often detrimental rather than beneficial.

(b) Shear strength of the soil is increased. The removal of water in cohesive soils increases its strength, which otherwise have almost negligible shear strength in a saturated condition.

(c) There is reduction in excess hydrostatic pressure. Removal of subsurface water reduces any excess hydrostatic pressure that may develop. Excess hydrostatic pressure is associated with a loss in normal forces and hence a loss in frictional shear strength. Thus subsurface drainage would tend to restore or increase the frictional shear strength of the soil mass.

Investigation

A field review of the site should be made by competent engineers or geologists who are familiar with the causes of slides and slipouts and the various methods of evaluating these causes. They should also have a knowledge of various techniques of correction that might be used for landslide correction.

The field review should then be followed up by geologic investigations and/or exploratory borings, either vertical or horizontal. The exploration should also provide information on ground water conditions.

Planning of Installation

The location and depth of ground water together with topography will determine the locations from which the drains will be started. Since the drains remove water by gravity, the starting point for a drain must be below the elevation of the point where the water is to be intercepted. The spacing of the drains depends on the drainage characteristics of the soil, the quantity of water intercepted, and the character and magnitude of the slide involved. Usually drains are planned at intervals of 25 to 100 ft. Drains are often installed from more than one level if the terrain permits and the distances are such that the subsurface water can be reached from various levels.

Depths to which drains are placed can vary between 50 and 300 ft. The average figures for depth of penetration vary between 100 and 200 ft. This depth is usually controlled by the depth to which the drains will have to extend to contact the water-bearing strata and to properly drain the area and produce a stable condition in the slide. Other factors involved are difficulty of drilling, quantity of water drained, the economy versus effectiveness of a greater number of shorter drains as compared with fewer but longer drains etc.

The drains are usually installed on grades ranging from 3 to 20 per cent, 10 per cent being the best working grade.

A system of collection pipes should also be provided to carry intercepted water out of the critical area. If the outlet of the drains discharges into an existing gutter drain, then no other arrangements are necessary. If such a roadside drain is not readily available, 6 to 8 in. galvanised corrugated metal pipes can be used to carry out the drain water to any desired location outside the slide area.

Equipments

The equipment, see Fig. 42, consists of a rotary drill, which is capable of advancing a drill bit into the slope. Standard perforated 2 in. iron pipe with the following specification is used for casing.

"Standard 2 in. black steel pipe perforated with $\frac{3}{8}$ in. diameter holes on 3 in. spacing drilled in 3 rows at quarter points, to be furnished in random lengths of 16 to 24 ft. without threads or couplings. Pipe to be vertically dipped in a standard pipe dipping asphalt subsequent to drilling." 175

Maintenance of horizontal drain installations is necessary if they are to continue to be effective for long periods of time. It is important that they should be cleaned at intervals of 3 to 10 years to remove root growths, corrosion and soil from drains.

Effectiveness

Drain installations are most effective in areas where subsurface water could be intercepted in well defined aquifers or layers, where the soil is sufficiently permeable to permit ready removal of water and where the water could be reached with holes not more than 300 ft. long on 5 to 15 per cent grades through formations that can be drilled successfully and where the borings do not cave in.

Two types of formations in which a successful installation is rather difficult to make are (i) silty fine sands, and (2) hard broken formations. In the silty fine sands, caving presents a major difficulty and in the hard broken formations, it is difficult to drill and there is loss of circulating fluid during drilling and casing operations.

The quantity of water that is produced at the time of installation is not necessarily a good index of the flow that will occur later or of the effectiveness of the installation. Some drains may produce large flows during the rainy season or during and after actual periods of rainfall, and remain dry or produce little water at other times. Other drains may produce flows that vary somewhat with the seasons. It should be noted that in some instances the removal of a small quantity of subsurface water will produce a stable soil condition whereas other instances may require the removal of very large quantities of water to produce the desired results.

Conclusions

Horizontal drains have a definite place in the correction and prevention of slides in embankments and in cuts and when properly planned, installed and maintained they are effective. The C.R.R.I. has conducted field trials to gain first-hand experience with such installations.

6.9 SLOPE TREATMENT BY ASPHALT MULCH TECHNIQUE OF VEGETATIVE TURFING SPONSORED BY C.R.R.I.

The Central Road Research Institute, on the basis of recent field trials, has found the asphalt mulch technique effective in the control of erosion on hillslopes by providing suitable vegetative turfing. This method of slope treatment is considered promising especially as an adjunct to be used in combination with other techniques of prevention or correction.

Brief details of the technique and the reasoning behind it are given below.

Firstly, the slope area proposed to be treated is demarcated and fenced by local prickly bushes or barbed wiring. The slopes are then prepared into vast seed beds by rounding off the tops, regrading or reshaping and by finally raking the top soil about 1 in. thick. If the slopes are entirely raw and infertile and if the soil happens to be slightly acidic, (as in Northern U.P.), calcium ammonium nitrate is applied at the rate of 100 lbs. per 5000 sq. ft. The root slips of the most promising types of locally available grasses are dibbled, 6 to 9 in. apart, root to root, and row to row, taking care to see that no turfs or clumbs are dibbled. An asphalt mulch of a specified grade is then spread by a suitable sprayer. Fig. 43 shows the spraying operation. The sprayer should be capable of easy handling, and one that is developed especially for the purpose, enabling quick treatment to be undertaken over vast areas. The optimum rate of application of the emulsion shall be 0.15 to 0.20 gal. per sq. yard. The thickness of the emulsion coating is to be an optimum, because thicker application would tend to retard the growth of plants and seeds, whereas applications thinner than optimum would not be effective in controlling erosion.

The advantages resulting from the application of asphalt are (i) susceptibility to erosion is cut down, (ii) the moisture content as well as the nutrients in the soil mantle are conserved, (iii) the soil temperature is raised by absorbing the light rays, promoting the emergence of tiny saplings.

The asphaltic film gradually disintegrates, its place being gradually taken up by a carpet of green vegetation. The carpet of grass, that supplants the asphaltic film, acts as an immediate cover for the slopes till the more deep-rooted species of shrubs and trees, develop and take root.

The method proves particularly successful if it is so timed that advantage is taken of the increased moisture content in the soil resulting from the first couple of monsoon showers. However, neither continuous heavy downpour nor a long spell of dry weather occurring immediately after the completed treatment is desirable since in such an eventuality, the process might perhaps have to be repeated partially or fully.

A comparison between Fig. 44 showing the untreated slope and Fig. 45 showing the same slope after treatment speaks for itself as far as the benefits of asphalt mulch technique are concerned. Fig. 46 shows a slope partly treated and partly untreated.

Equipment Required for Asphalt Mulch Technique

(I) Sprayers (Fig. 47)

(a) Knapsack sprayer. 2 to 4 gals. capacities. Wt. 6½ to 16 lbs. Being portable, it is particularly suitable for steep slopes above the highway.
(b) Gator rocking sprayer. Wt. 20 to 40 lbs. Capacity of pressure upto 200 psi. Can be used on upslopes with the provision of ramps.
(c) Hand compression sprayers. Capacity 3 to 4 gals. Wt. 13 to 20 lbs.
(d) Charge pump with spray tanks.

(II) Spray guns and lances and boom.

(III) Spray nozzles
(a) Three action adjustable nozzle.
(b) Circular mist type nozzle.
(c) Flat fan type nozzle.

(IV) Hose connections and pressure plastic tubing ½ in. or suiting to hose connection. Enough length with connectors.

(V) 12 in. adjustable wrench, screw driver and a plier.

(VI) Strainer 30 to 50 mesh.

(VII) Kerosene.

(VIII) Bitumen emulsion or cut backs.

(IX) Rope ladders and other equipment for facilitating spraying job on steep high-up slopes.

(X) Cotton waste.

(XI) Drum opener.

(XII) Buckets and trays.

The equipment should be cleaned thoroughly with kerosene or some other solvent at the close of each day's work. Failure to do so would result in clogging and jamming of the equipment thereby making delayed cleaning somewhat more difficult. After the bituminous emulsion is filled in the container, it is pressurised by means of a pump upto 60 to 70 psi and maintained constant subsequently. The use of higher pressure is avoided because higher pressures generally give rise to uneven spray pattern.

6.10 SLOPE TREATMENT FOR EROSION CONTROL USING JUTE MESH OR NETTING

It has been found that if a heavy mesh of jute fabric is firmly laid on loose earth and sown with suitable grass seeds, it gives maximum protection to the soil until the grass takes root and furnishes a permanent coverage. After the soil is stabilised, the nettings decompose and provide nourishment to the grass growing on a soil medium which hardly possesses any nutrient. A typical netting has one in. openings between the threads giving the grass plenty of room to grow and at the same time providing a large number of 'check dams' per sq. yard

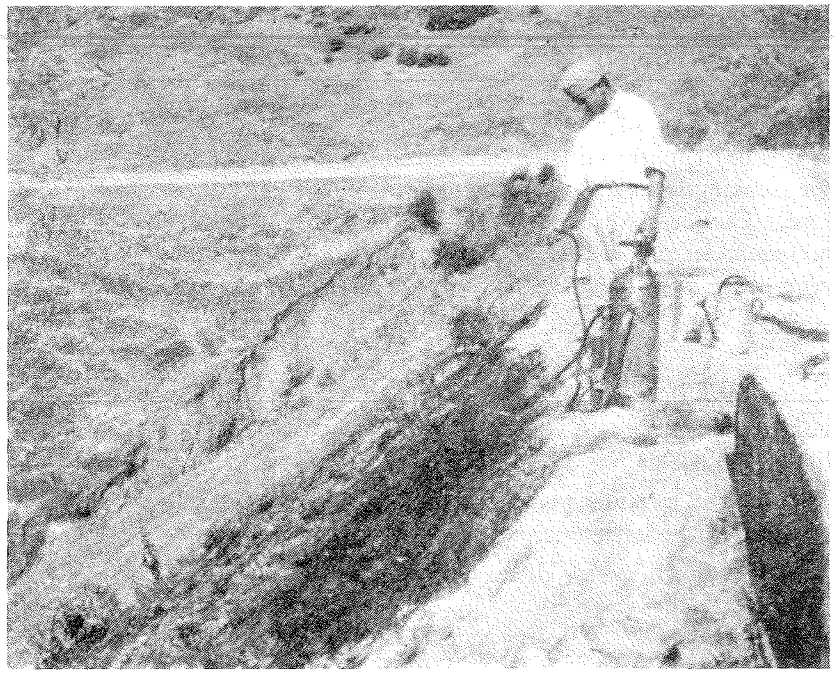
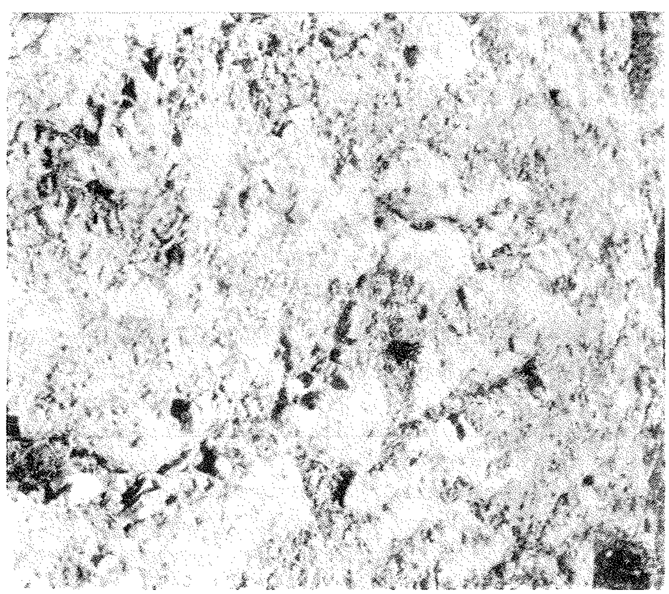


Fig. 43 Spraying of bituminous emulsion

Fig. 44 Slope before treatment (asphalt mulch technique)



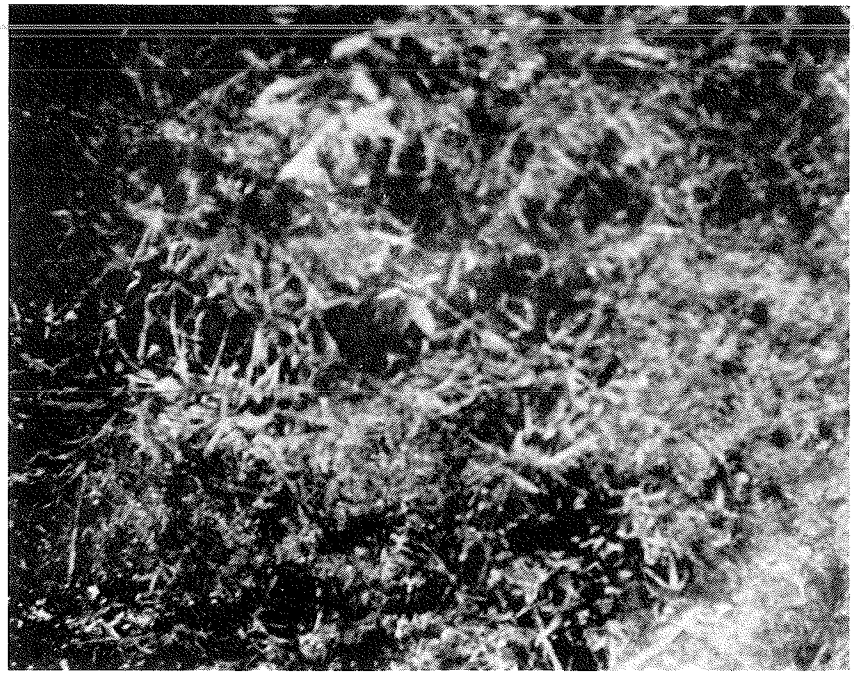
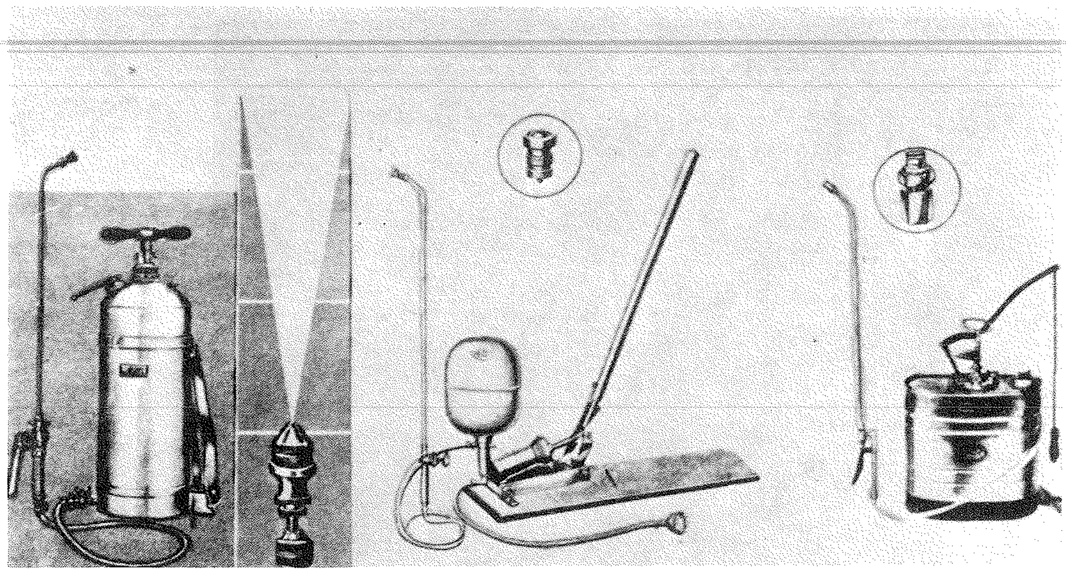


Fig. 45 Condition of slope after treatment (asphalt mulch technique)

Fig. 46 Slope partly treated and partly untreated



(a) (b) (c)

Fig. 47 (a) Hand compression sprayer with 3-action adjustable nozzle (b) Gator rocking sprayer with circular mist type nozzle (c) Knapsack sprayer with flat fan type nozzle

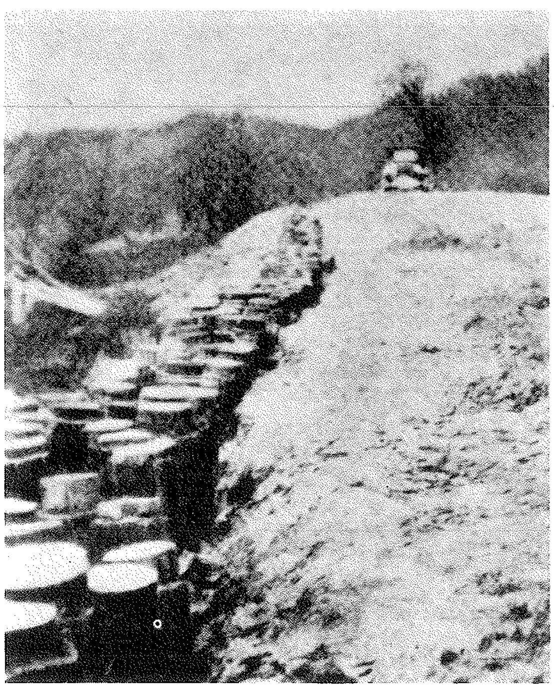


Fig. 48 Timber piles as a protection to the road

Fig. 49 Steel rails used to hold a slide

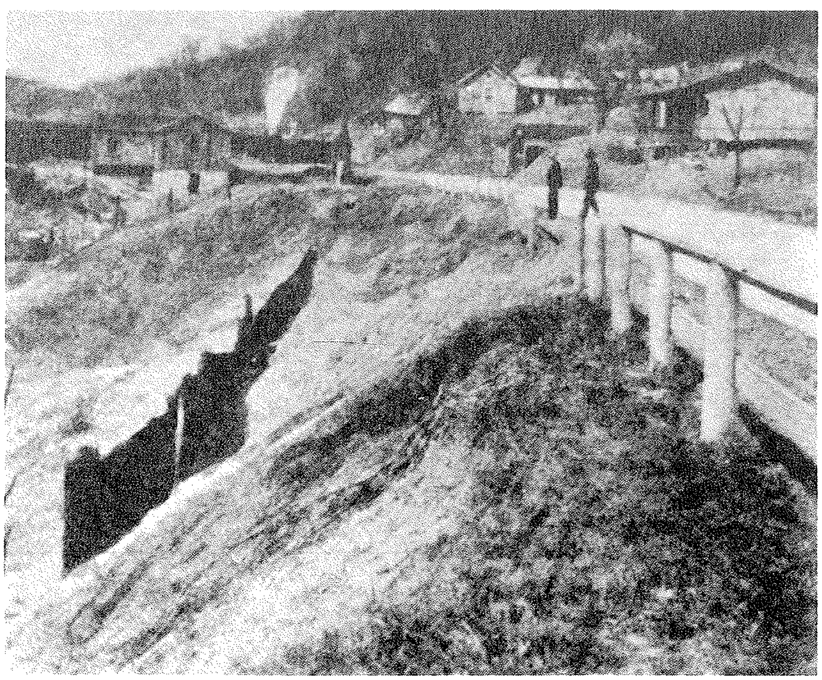


Fig. 50 Steel sheet piling to hold the embankment

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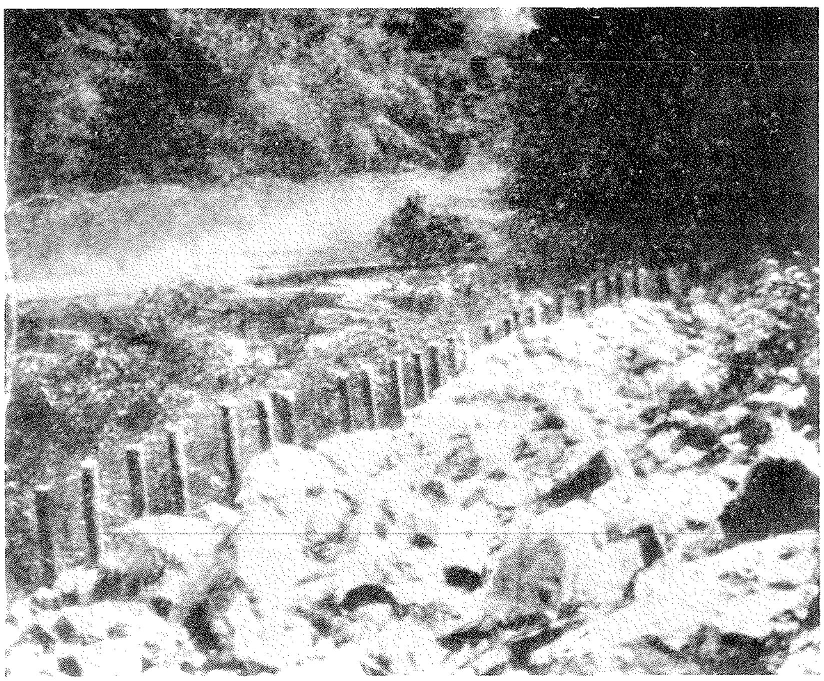

Fig. 51 Boulders placed at toe of sliding mass to supplement piling

of the material. The nettings are available in rolls, 48 in. wide. The netting is just rolled out on the area to be treated and is properly secured over the ground where a concentrated flow of water is liable to occur and where there is danger of undercutting of the soil. The netting is firmly secured on to the sloping surface by means of special staples at specified intervals. Control of erosion is effected immediately after the netting is laid. One half of the normal rate of seeding is broadcast prior to the placement of the netting and the other half is broadcast after the netting is in place.

The Central Road Research Institute in collaboration with the Indian Jute Industries Research Association, Calcutta have conducted field trials using such jute scrims for erosion control and the results have been very promising. It is generally believed that the use of jute netting for erosion control may be resorted to especially where the erosion problem is very serious and the kinetic energy of the flowing water is considerable and where other conventional methods of erosion control prove to be of no avail. For further details concerning specifications, enquiries may be addressed to the Central Road Research Institute or the Indian Jute Industries Research Association, Taratola Road, Calcutta 53.

6.11 USE OF TIMBER PILES

Timber piles can be used to stabilise small potential slides.

In order to stabilise a potential slide, the timber piles are driven into the ground at the edge or shoulder of a cut. Fig. 48 shows closely spaced piling to hold the road in place. In effect, the driving of piles helps to compact the mass especially in sandy or silty soils and thus increase the shearing resistance in the potential slide area.

More than one row of these timber piles may be required to be driven depending on the extent of the slide mass to be controlled. For best results, the piles should be driven to a firm soil strata, such as firm shale. Piles should not, of course, be driven in soil which may liquefy due to vibration. The timber used for piles can be square or round and of good quality, preferably locally available. Untreated timber piles can be used, but it should be noted that their life is short. Timber piles treated with a preservative such as creosote last longer.

In any given slide area, the chances of timber piles becoming ineffective because of the potential failure surface passing below the tip of the piles must be guarded against. This may require a stability analysis to be carried out. Piles may become ineffective if a flow occurs, the soil flowing out around and between the piles. Other possible modes of failure are (i) the piles themselves may fail in shear, (ii) piles may fail by overturning.

Rows of steel rails, as shown in Fig. 49, have occasionally been used for control of slides. Attempts have also been made to use sheet piling to retain the fill as illustrated by Fig. 50. Sometimes a load of boulders is used to supplement piling. The boulders massed along the toe of a fill as shown in Fig. 51, probably do exert some restraining effect, but on the whole their use is of little value.

6.12 BLASTING

The basic principle of blasting, in simplest terms, is to disrupt a volume of virgin rock in such a way that the desired stability of the slope is achieved. Preparation for the blast is a vital factor which must be meticulously planned with the fullest regard to all the known characteristics of the rock to be removed. Details of drilling pattern, depth of holes, quantity of explosives, sequence of detonation etc., must be worked out for the success of the work.

As a general principle, charges should be so placed that the line of least resistance to the thrust will be towards a free or open face.

The benefits of blasting as a method for slope correction is rather controversial. According to one school of thought 'blasting' is no solution to produce a long range correction while the opposing school of thought believes that many an installation have given satisfactory performance for many years.

The benefits of blasting may be listed as follows.

(1) It can produce better drainage beneath the surface of rupture thereby providing a release to the excess hydrostatic pressure.

(2) It can disrupt and relocate the critical surface of sliding. This relocation of slip surface in changed soil conditions improves the shearing resistance.

Blasting as a method of slope correction is generally recommended where slide failures are imminent. For flow types of movement, it is considered not suitable. The method is limited by operational and long range economic factors to masses of less than 50,000 cubic yards in size.

The reason for preference of this method over the others is the initial economy of installation. Experience with blasting and other methods of slope correction has been that where applicable, the former will usually cost one fifth to one tenth of the cost of the latter. It should be noted that a couple of blasts over a period of several years can be more economical than a single correction by other possible methods.

The points against blasting are unpredictability of success and the possibilities of damage due to the dynamic effects of the blast and the dangers of overshooting. It should be remembered that this method cannot be a success unless a relatively firm bed rock underlies the surface of rupture.

Chapter 8 BASIC RULES ON ANALYSIS FOR (1) PREVENTION, AND (2) CORRECTION OF LANDSLIDES

Some of the basic rules to be followed concerning the steps that are to be taken to prevent or control potential or incipient landslides whenever hill roads have to pass through troublesome ground and also the steps that are to be taken in correcting a landslide that has already occured are set forth in this Chapter. The rules, more or less, represent an outline of a step-by-step procedure which can be usefully followed for purposes of field investigation of actual landslides. The rules have been formulated solely from the criterion of either reducing landslide susceptibility or correcting actual landslides. As such, general rules pertaining to alignment, grade, design, speed on hill roads etc., are considered to fall outside the scope of the Handbook and are not included in this Chapter.

A. Rules Relating to Location of New Lines of Transportation in Hills from the View Point of Landslide Prevention

(1) In locating new lines of transportation, avoid 'troublesome' ground. The following types of soil constitute troublesome ground and should be avoided as far as possible:

(a) Sand below water table.
(b) Homogeneous soft clay formations.
(c) Stiff fissured clays like over-consolidated clay shales.
(d) Loess or loessial formations.

(2) Certain types of landforms such as those listed below in which landslides are most susceptible should be avoided as far as possible:

(i) Basaltic lava.
(ii) Serpentine.
(iii) Soft clay shales.
(iv) Tilted sedimentary rock.
(v) Weathered rocks containing chlorite-talc or mica-schist.
(vi) Detritus and talus materials.

(3) The construction of cuts in favourable ground, such as cohesive, sandy or cohesionless soil in a moist or dry state, is a fairly well standardised procedure. Experience shows that slopes of $1\frac{1}{2}$ horizontal to 1 vertical, are commonly stable. As a matter of fact, the height of some of the highway cuts less than 20 ft. rise at that slope, as do the height of many deeper stable cuts. Therefore, for a cut, a slope of $1\frac{1}{2}$ horizontal to 1 vertical, is considered the standard for highway construction. The standard slopes for flooded cuts such as those for canals, range between 2:1 and 3:1. Steeper than standard slopes should be established only on rock and dense sandy slopes interspersed with boulders, and on true loess soils.

(4) If the cut is to be made in troublesome ground, the engineer responsible for the alignment should consider the combined soil and hydraulic conditions that are obtaining along the proposed alignment, so that he may make the required departure from the customary standards of safety. The engineer-in-charge of the location must be capable of identi-

fying favourable, troublesome and very troublesome ground on the basis of surface indications and occasional borings. He must also be able to visualize the worst construction difficulties that may arise at the various places and to evaluate the corresponding expenses or delays.

(5) If troublesome ground cannot be avoided, the following tasks must be performed:

(a) Locate the most critical site and explore the site by sampling and testing.

(b) Select the slope angles on the basis of a reasonable compromise between economy and safety.

(c) Design the drainage system (such as road side ditches, catch water drains, drop channels and cross drainage works) whenever necessary.

(d) Programme the observations that must be made during construction to remove the uncertainties in the knowledge of the site conditions and eliminate the risk of accidents.

(e) Stabilise those slopes that begin to move out at a minimum of expense and delay.

Whenever the analysis of the landslide does not lend itself to simple classification on the basis of visual observations and whenever the landslide defies correction on the basis of simple corrective measures, the problem may be referred to an expert or a research organisation dealing with the subject.

(6) By means of a personal visit and inspection of the hill slopes, try to identify and recognise potential or incipient landslides by watching out for evidences of 'stretching' of the ground surface. 'Stretching' is distinguished from 'soil creep' inasmuch as it indicates a comparatively deep-seated movement, whereas 'soil creep' is of superficial origin. 186 Usually, stretching is most commonly observed in relatively cohesionless materials that cannot form or retain minor cracks readily. One of the prominent evidences of stretching would consist of small cracks that surround or touch some rigid body, such as a root or a boulder, in an otherwise homogeneous material. These cracks form because the tensional forces tend to concentrate at or near the rigid bodies. Evidences of creep can be had from the fact that certain trees have got tilted invariably toward the direction of soil creep.

(7) Look out for visible signs of ground movement such as a settlement of the highway or depending on the road's location within the moving mass, an upbulge of the pavement. In some cases, it is possible to find evidences of landslide movement that has not yet affected a highway but that may do so any time. Thus, minor failure in the embankment, materials that fall on the roadway from the upper slope or even the progressive failure of the region below a fill may well presage a larger landslide that will endanger the road itself.

(8) Collect other evidences of movement such as can be found in broken pipe or power lines, spalling or other signs of distress in concrete structures, closure of expansion joints in bridge plates or rigid pavements, or loss of alignment of building foundations. In many cases, arcuate cracks and minor scarps in the soil give advance notice of serious failure.

(9) Cultivate the ability to recognise small cracks and displacements in the surface soils and understand their significance. Understanding the meaning of cracks is a faculty which can help produce an accurate knowledge of the causes and character of movement that is a prerequisite to correction. Small 'en echelon' cracks commonly develop in the surface soil before other signs of rupture take place, Fig 27. They are thus particularly valuable tools in the recognition of potential or incipient slides. In many cases, a map of en-echelon cracks will delineate the slide accurately, even though no other visible movement has taken place.

(10) Making use of the data collected and by studying the cracks in the surface soil (see Chapter 5) determine the types of slide with which you are dealing. For example in a slump, the walls of cracks are slightly curved in the vertical plane and are concave towards the direction of movement; if the rotating slump block has an appreciable vertical offset the curved cracks wedge shut in depth. In block glides, on the other hand, the cracks are nearly equal in width from top to bottom and do not wedge out in depth. This is because, failure in block glides begins with tension at the base of the block and progresses upward towards the surface. Block glide can be distinguished from lateral spreading by the presence of a few major breaks in the upper parts of a block glide whereas lateral spreading is characterised by a maze of intersecting cracks.

Cracks in block glides in cohesive soils are commonly almost vertical, regardless of the dip of the slip-plane, whereas in block glides of rock, the inclination of the cracks depend on the joint systems in the rock.

One of the most helpful applications of the study of cracks, lies in the distinction between incipient block glides and slumps. If the outline of the crack pattern is horseshoe—shaped in plan, with or without concentric cracks within it, a slump is almost certainly indicated. If, on the other hand, most of the surface cracks are essentially parallel to the slope or cliff face, a block glide is probably in the making. In either case, additional cracks may develop as major movement gets under way, but these will generally conform to the earlier crack pattern.

(11) Using Table 8 as a guide, identify the type of landslide. Table 8 summarises the surface features of the various parts of a slide and could serve as a guide for the indentificaion of different landslide types. 187

(12) Once the fact of land movement has been established, the next essential step is to identify the type of landslide as suggested in rule 11 above. Having identified the landslide type, make an appropriate choice of the preventive measures on the basis of reasoning given in Chapter 6.

(13) For purposes of slope design in soil cuts and in bed cuts, please see Chapter 2 and 3 respectively.

(14) If the formation through which the road passes constitutes troublesome ground such as detritus, loose sand, soft homogeneous clay, stiff fissured clay or loess, please see Chapter 4 and Chapter 6 for further information.

The following represent other basic rules concerning alignment and drainage in hill roads passing through areas of potential landslides:

(15) Where the proposed excavation will cross formations that are susceptible to bedding plane slides, the slide hazard can sometimes be reduced by adjusting the alignment so that the cut slopes intercept the beds at a more favourable angle to the bedding planes. The alignment should be so adjusted that the bedding plane of the rock tends to dip away from the cut slopes rather than towards them.

(16) In locating an alignment, consideration should be given to some of the typical situations conducive to landsliding, induced by proposed cuts or fills. These are:

(a) Restriction of ground water flow by side hill fill.

(b) Overloading of relatively weak underlying soil layer by fill.

(c) Overloading of sloping bedding planes by heavy sidehill fill.

(d) Oversteepening of cuts in unstable rock or fill.

TABLE 8

The following represent a description of the features that will aid recognition of active or recently active landslides.

1. FALL

(i) Rockfall

Crown
Loose rock: probable cracks behind scarp; irregular shape controlled by local joint system.
Main Scarp
Usually almost vertical: irregular, bare, fresh. Usually consists of joint or fault surfaces.
Flanks
Mostly bare edges of rock.
Head
Usually no well-defined head. Fallen material forms a heap of rock next to scarp.
Body
Irregular surface of jumbled rock, sloping away from scarp. If very large and if trees or material of contrasting colour are included, the material may show direction of movement radial from scarp. May contain depressions.
Foot
Foot commonly burried. If visible, the foot generally shows evidence of reason for failure, such as underlying weak rock or banks undercut by water.
Toe
Irregular pile of debris or talus if small. If the rockfall is large, the toe may have a rounded outline and consist of a broad, curved transverse ridge.

188 **(ii) Soilfall**

Crown
Cracks behind scarp.
Main Scarp
Nearly vertical, fresh, active spalling on surface.
Flanks
Often nearly vertical.
Head
Usually no well-defined head. Fallen material forms a heap next to scarp.
Body
Irregular.
Foot
Same features as per 'Rockfall' described above.
Toe
Irregular.

2. SLIDES

(i) Slump (Soil)

Crown
Numerous cracks, most of them curved concave toward slide.
Main Scarp
Steep, bare, concave toward slide, commonly high. May show striae and furrows on surface running from crown to head. Upper part of scarp may be vertical.
Flanks
Striae on flank scarps have strong vertical component near head, strong horizontal component near foot. Height of flank scarp decreases toward foot. Flank of slide may be higher than original ground surface between foot and toe. En echelon cracks outline slide in early states.
Head
Remnants of land surface flatter than original slope or even tilted into hill-creating depressions at foot of

main scarp in which perimeter ponds form. Transverse cracks, minor scarps, grabens, and fault blocks may be noticeable. Attitude of bedding differs from surrounding area. Trees lean uphill.

Body

Original slump blocks generally broken into smaller masses: longitudinal cracks, pressure ridges, occasional overthrusting. Commonly develops a small pond just above foot.

Foot

Transverse pressure ridges and cracks commonly developed over the foot: zone of uplift, absence of large individual blocks, trees lean downhill.

Toe

Often a zone of earthflow, lobate form, material rolled over and buried: trees lie flat or at various angles mixed into toe material.

(ii) **Slump (Rock)**

Crown

Cracks tend to follow fracture pattern in original rock.

Main Scarp

Same features as per 'Slump (Soil)' described above

Flanks

Same features as per 'Slump (Soil)' described above.

Head

Same features as per 'Slump (Soil)' described above.

Body

Same features as per 'Slump (Soil)' described above, but material does not break up as much or deform plastically.

Foot

Same features as per 'Slump (Soil)' described above.

Toe

Little or no earthflow: toe often nearly straight and close to foot: toe may have steep front. 189

(iii) **Block Glide (Rock or Soil)**

Crown

Most cracks are nearly vertical and tend to follow contour of slope.

Main Scarp

Nearly vertical in upper part, nearly plane and gently to steeply inclined in lower part.

Flanks

Flank scarps very low, cracks vertical. Flank cracks usually diverge downhill.

Head

Relatively undisturbed. No rotation.

Body

Body usually composed of a single or a few units, undisturbed except for common tension cracks. Cracks show little or no vertical displacement.

Foot

No foot, no zone of uplift.

Toe

Plowing or overriding of ground surface.

(iv) **Rockslide**

Crown

Loose rock, cracks between blocks.

Main Scarp

Usually stepped according to the spacing of joints or bedding planes. Surface irregular in upper part, and gently to steeply inclined in lower part; may be nearly planar or composed of rock chutes.

Flanks

Irregular.

Head

Many blocks of rock.

Body
Rough surface of many blocks. Some blocks may be in approximately their original attitude, but lower down, if movement was slow translation.
Foot
Usually no true foot.
Toe
Accumulation of rock fragments.

3. FLOWS

(i) **Rock Fragment Flow (Dry)**
Crown
Same freatures as per 'Rockfall' described above.
Main Scarp
Same features as per 'Rockfall' described above.
Flanks
Same features as per 'Rockfall' described above.
Head
No head.
Body
Irregular surface of jumbled rock fragments sloping down from source region and generally extending for out on valley floor. Shows lobate transverse ridge and valleys.
Foot
No foot.
Toe
Composed of tongues. May override low ridges in valley.

190 (ii) **Sand Run (Dry)**
Crown
No cracks.
Main Scarp
Funnel-shaped at angle of repose.
Flanks
Continuous curve into main scarp.
Head
Usually no head.
Body
Conical heap of sand, equal in volume to head region.
Foot
No foot.

(iii) **Debris Avalanche or Debris Flow (Wet)**
Crown
Few cracks,
Main Scarp
Upper part typically serrate or V-shaped. Long and narrow bare, commonly striated.
Flanks
Steep, irregular in upper part. Levees may be built up along lower parts of flanks.
Head
May be no head.
Body
Wet to very wet. Large blocks may be pushed along in a matrix of finer material. Flow lines. Follows drainage lines and can make sharp turns. Very long compared to breadth.
Foot
Foot absent or buried in debris.

Toe

Spreads laterally in lobes. Dry toe may have a steep front a few feet high.

(iv) **Earthflow (Wet)**

Crown

May be a few cracks.

Main Scarp

Concave toward slide. In some types, scarp is nearly circular, slide issuing through a narrow orifice.

Flanks

Curved, steep sides.

Head

Commonly consists of a slump block.

Body

Broken into many small pieces. Wet. Shows flow structure

Foot

No foot.

Toe

Spreading, lobate. See above under 'Slump'.

(v) **Sand or Silt Flow (Wet)**

Crown

Few cracks.

Main Scarp

Steep, concave toward slide, may be variety of shapes in outline—nearly straight, gentle arc, circular, or bottle-shaped.

Flanks

Commonly flanks converge in direction of movement.

Head

Generally under water. 191

Body

Spreads out on underwater floor.

Foot

No foot.

Toe

Spreading, lobate.

(e) Removal by cuts of thick mantle or pervious soil if such pervious soil happens to be a natural restraining blanket over a softer core.

(f) Increase in seepage pressure caused by cut or fill that changes direction and character of ground water flow.

(g) Exposure by cut of stiff fissured clay that is liable to soften and swell when exposed to surface water.

(h) Removal of mantle of wet soil by sidehill cut. Such a cut may remove toe support causing soil above cut to slide along its contact with stable bed rock.

(i) Increase in hydrostatic pressure below surface of a cut in silt or permeable soils.

(17) The surface drainage of steep hills should be meticulously planned so that the potential danger of landslide by the high velocity of flow in storm periods is obviated, ensuring roads safety. It is customary to have a drain on either side when a road traverses through a plain country but on hills, normally, there should be two drains on one side only—one along the side of the road to drain off water from the road surface and the other higher up

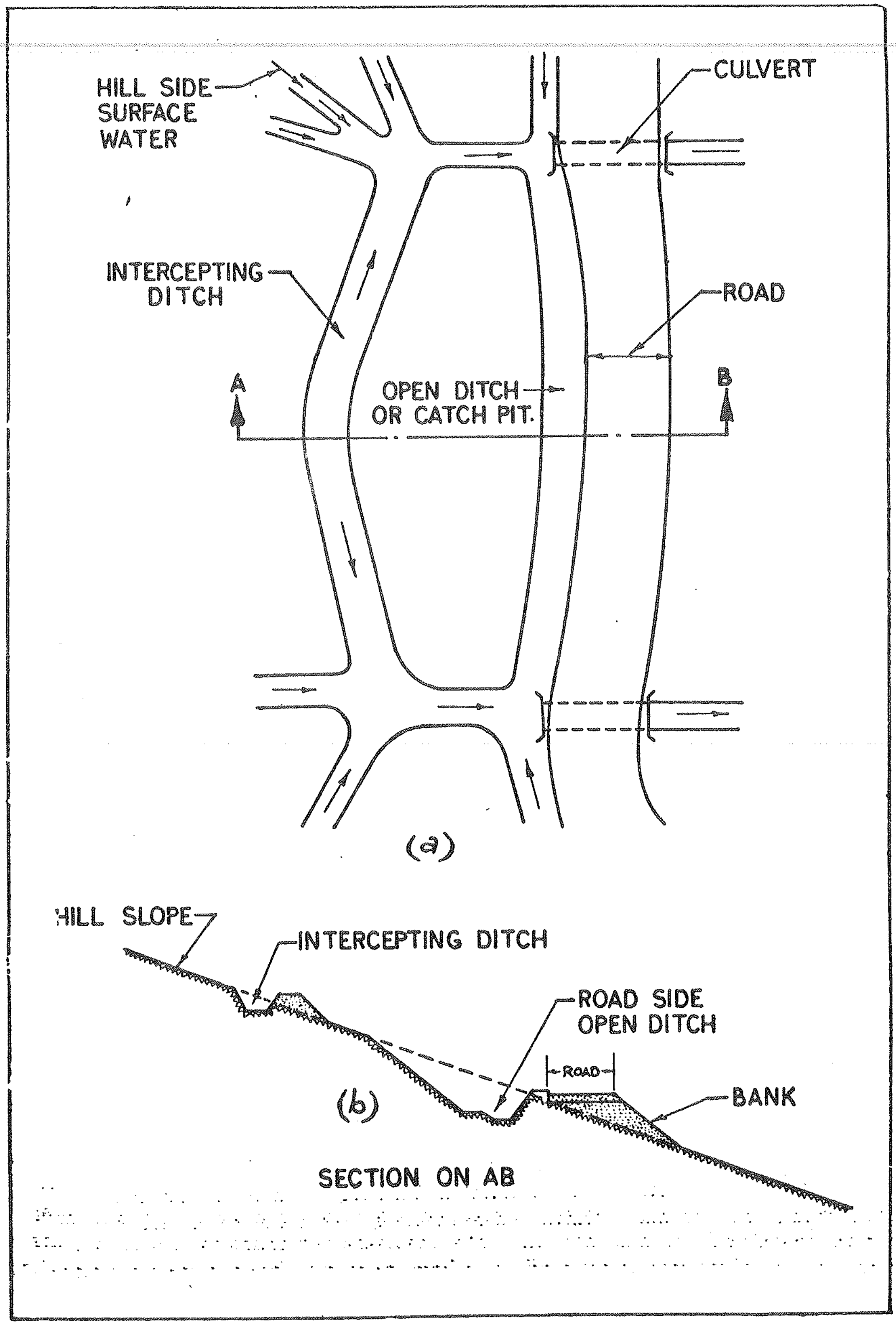

Fig. 57 A typical drainage system

the hill in order to intercept the run-off from the hill slope and draining into the culverts. Fig. 57 shows the plan and the section of a typical drainage system.

(18) Catch water drains require special attention and on ocassions it may be necessary to build two or more catch water drains, one above the other, on the hill side discharging into a main drain, leading directly to culverts or to one of the adjoining drainage crossings on the road. A catch water drain of suitable cross-section, preferably lined along its wetted perimeter, laid at a longitudinal gradient of say 1 in 50, helps considerably towards surface drainage.

(19) When the road traverses through a hilly country, the width available is restricted and as such any drain placed by the side of the road is bound to reduce the available effective width of the road. It is, therefore, desirable to build the drain in such a way that besides functioning as a drain it may also act as a part of the road surface in an emergency when fast moving vehicles are compelled to move to the extreme edge of the road to aviod road accidents. To attain this objective, 'angle', 'saucer' or 'kerb' and 'channel' drains would be helpful, Fig. 58.

(20) The efficacy of catch water drains is somewhat questionable unless they are lined with bitumen or cement plaster in order to minimise percolation of storm water but lining in most cases may become impracticable for want of funds.

(21) For unimportant roads, stone or timber culverts, built of locally available stone or timber may prove adequate.

(22) To prevent a shallow road side drain from flooding the road surface, culverts or cross-drains should be provided at suitable intervals thereby guiding the discharge from the road side down to the valley. 193

(23) For proper maintenance of the drainage system of the hill road, the drains and culverts should be kept perfectly clean before the monsoon sets in and later on inspected after each shower of rain. Any blockage of the drain should be cleared off immediately.

(24) Whenever frequent choking of drains is inevitable, tile drains may be used. These consist of porous fire clay pipes placed in trenches and surrounded with stones and sand. This material should be suitably graded and, if need be, placed in layers of decreasing grain sizes. The aim is to present a minimum of resistance to the passage of water, and at the same time to prevent soil particles from being washed through into the drain.

(25) It is generally preferable to take a road as much as practicable along the ridge of the hill with a view to limit the number of drainage crossings to the minimum. Fig. 59 clearly illustrates that an alignment ABC should be preferred to the alignment AC for the only reason that alignment ABC avoids the requirements of drainage although the length of the road in this case will be somewhat longer.

(26) In cuttings through unfavourable ground, the hill sides get freshly exposed and are rendered particularly susceptible to serious erosion hazards. In fact, any soil slope denuded of vegetation should be regarded positively as a potential seat of landslide. In all these cases, vegetative turfing using the locally available species of grass and deep-rooted plants should be resorted to. Invariably, the appropriate time for taking up the work of seeding the slopes is just prior to the onset of monsoon or alternatively at the time of withdrawal of monsoon. However, in the case of steep slopes and in situations in which the erosion problem is serious, all the painstaking effort involved in the seeding of the slopes would prove futile if the first couple of monsoon showers should wash away the seeds which have been broadcast. Furthermore, the job of turfing cannot normally be undertaken during any other season

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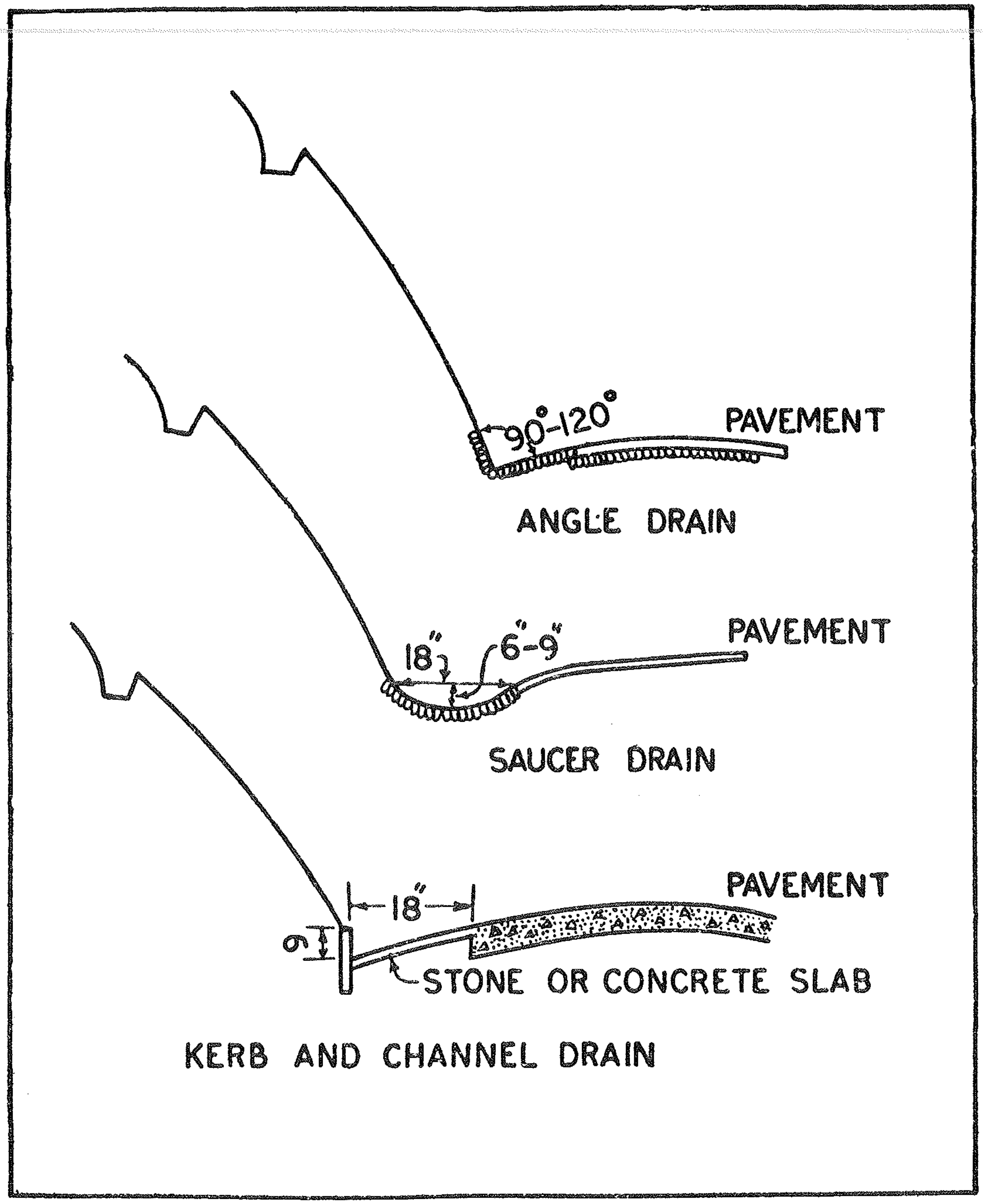

Fig. 58 Types of drains

due to the inadequate moisture content of the soil slopes. In order to obviate the risk of erosion taking place immediately after seeding has been done, the asphalt mulch technique should be resorted to. The details and principles of the technique are given in Article 6.9.

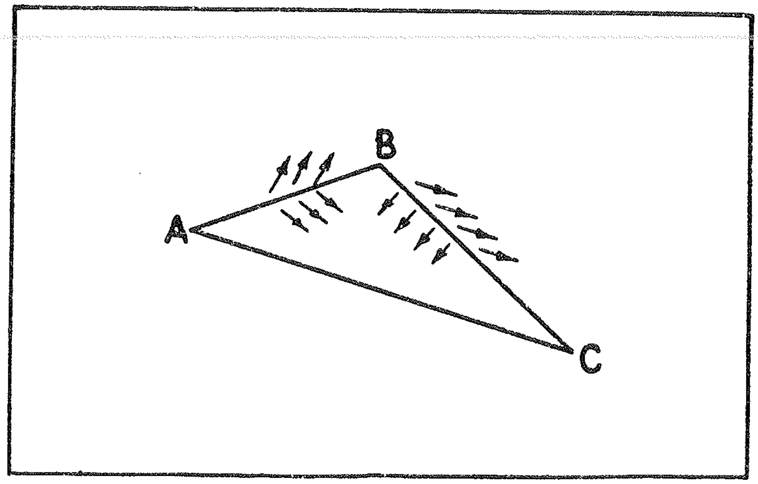

Fig. 59 Alignment ABC should be preferred to alignment AC

Under particular site and climatic situations, vegetative turfing without the use of asphalt mulch can be resorted to, provided it is ensured that (i) the moisture conditions under the treated slopes are favourable for the germination of the seeds, and (ii) the risk of erosion or washing away of the seeds is not imminent.

B. Rules Relating to Field Investigation of Actual Landslides with a View to Planning Control and Corrective Measures 195

The following rules are meant to guide the engineer as to the steps he should take and the manner in which he should go about with regard to field investigation when a landslide has actually occurred, with a view to planning the appropriate measures for repair and reconstruction.

(1) One of the foremost and important steps in the field investigation of an actual landslide is to visit the affected area and to bring oneself to climb up the hill slope so as to look out for visual evidences of the character of movement and the probable causes of the slide. The investigator should inspect the crown of the landslide and look out for the presence of cracks and fissures in the ground which will be of significance to him in the analysis of the landslide.

(2) The investigator, by means of a personal visit to the site, should obtain a knowledge of the general setting. By setting is meant all the factors that make up the physical environment—geology, soils topography etc. Accompanying such a procedure, he should try to obtain the necessary background knowledge by studying available aerial photographs and all topographic, geologic and soil maps of the locality, if possible. By a personal survey of the affected area combined with such facilities, the trained observer can obtain a great deal of information on the character of the slopes, of surface and subsurface drainage, and on the character and distribution of the different kinds of rock and soils that cover them. He should collect all relevant information and try to provide answers for the questions contained in the 'Questionnaire' appended to the Handbook. Most of the answers could be collected by a visit to the site of the landslide and by examining the physical conditions obtaining at the site, by local inquiries and from maintenance records.

(3) Study the site of the landslide from a distance, for the forest is more easily recognised than are the trees. Give special attention to the slopes, changes in slope and their relationship to the different materials involved.

(4) Map the landslide so as to obtain and record in graphic form such data as may be observed in the field from which significant inferences and facts relating to the cause, mechanics and potentialities of movement, past, present and future, may be drawn. The mapping procedure is classified into (a) general or areal, and (b) geographic. The purpose of the general or aerial mapping procedure is to fix the slide area in space so that there can be no doubt as to its geographic position. It is desirable, though not essential to locate the slide with reference to the mean sea level. For this purpose, some point near or associated with the slide area must be referenced to an acceptable benchmark. Such benchmarks may be available on bridge abutments or on easily recognised topographic features. For details, see Chapter 5.

(5) The map scale to be used is largely a function of the areal extent and economic importance of the slide in question, and in some cases it may also depend to some extent on the use to which the map is put. A small slope of only a few hundred feet, but involving extensive property damage or physical injury to the public may justify mapping on a scale such as 1 in. on the map to 5 or 10 ft. on the slide area. On the other hand, a slide covering several hundred or thousands of acres may be mapped on the scale of 1 in. on map to 50 or 100 ft. on the ground. However, some smaller portions of the same slide might be selected for mapping on a much larger and more revealing scale.

(6) It is often desirable to map not only the areal limits of the slide and the position of significant features within it, but also the physical configuration or topography. In this case, a contour map may be prepared. Here again, the judgement of the analyst must be exercised, first to decide whether a contour map is essential, and second, in the choice of the most desirable contour interval to illustrate the surface features of the slide. Whereas 2-feet contour intervals may be required in one slide study, a 10 or 20 or even 50 ft. contour interval may be satisfactory in another.

(7) The field methods employed in mapping the slide area are flexible and the choice would vary with the importance and degree of accuracy required. The accuracy of mapping becomes more important if continuing ground movement exists or is anticipated. In order to ensure high accuracy, triangulation stations should be established on stable ground, outside the slope area. From these, baseline points in the slide area may be established and checked periodically for movement.

(8) It may be desirable to survey and set up a grid system over the slide region. In such a case, the grid squares may be on 25 or 100 ft. centres, or any other distance that seems applicable to the problem at hand. The grid corners, once determined, may be used to check both horizontal and vertical movements. Sections, or topographic profiles, may be prepared along the grid lines, and overlays representing various time intervals often reveal striking changes in the slide surface that otherwise might go unnoticed. Regular checks may indicate changes in the movement rate such that resurgent acceleration may be indicated in time to forestall a catastrophe.

(9) Use either a plane table or regular surveying methods for preparing planimetric or contour maps and for determining the positions of reference points within and outside the slide area. Either method is applicable to mapping of small areas, but level and transit methods are perhaps to be preferred for larger ones. Accurate maps also can

be made, of course, by special methods from aerial or even terrestrial photographs. It is doubtful, however, whether such methods can be applied satisfactorily and economically to most ordinary landslide problems.

(10) Across the contours, or up and down the slopes, the following principles may be applied. The minimum distance upward should be atleast to the first sharp break in slope above the slide crown. The maximum distance needed would be upto the top of the slope. Intermediate distances can be chosen, depending on the physical features of the terrain and the judgement of the analyst. The minimum downward distances that the map should illustrate is to the first sharp break in the slope below the slide toe. The maximum downward distance is the bottom of the slope. Again, intermediate distances depend on the terrain and the analyst's judgement.

(11) The final map should show the slide proper, associated water conditions, and its geologic framework. The limits of the slide can be mapped first so as to depict its shape and size. The limits defined below are illustrated graphically at the bottom of Fig. 2 showing the 'Classification of Landslides.'

The upper part of the slide is the crown, or that point where the slide mass breaks away from the original ground slope. The cliff-like face below the crown is the main scarp. The contact of the mass of slide debris with the main scarp is the head of the slide. These together mark the upper limit of the slide. The lower limit of the slide is the toe, which is the margin of the disturbed material most distant from the main scarp. The tip is that point on the toe most distant from the crown of the slide, or the flanks. Displacement and the slide mass with reference to the crown and flanks should be mapped. Displacement at the toe may not be measurable because the foot (the line of intersection between the lower part of the surface of rupture and the original ground surface), may be buried. This displacement at the toe, however, may be inferred by interpolation and projection. Slopes on the main scarp below the crown and on the flanks should be determined, because they may aid in determining the depth and character of the slide mass. The surface of separation is the basal limit, or else the surface of rupture.

(12) The surface of rupture is easily recognisable at the crown and on the flanks, where it is the limit of displacement and where it may, in fact, be marked by a cliff or scarp. Underground, however, where it forms the bottom of the slide, no such striking expression calls attention to its presence and it can only be determined by means of subsurface exploration. In dealing with slides in which the slide mass and frame are composed of the same homogeneous materials, the recognition of the slide plane or surface of rupture may have to be based only on the striations or slickensides developed by motion of the slide mass on the 'plane of failure'. Commonly, this plane of failure is a series of closely spaced subparallel surfaces in and between which detrital rock fragments, if present, will be oriented in parallel with the plane of failure. The materials in this zone of failure are usually softer than in the overlying slide mass or in the underlying stable ground. The water content of the material in this zone is generally higher than in the disturbed and undisturbed materials above and below, because of higher permeability of the fractured material and in many cases, of a rather direct connection to a source of moisture. Relatively higher water content and lesser resistance to penetration may often prove more reliable indications of the location of the surface of rupture in borings and samples than are slickensides and striations. In many cases the surface of rupture is to be determined finally by correlating zones of high water content and low penetration resistance in several borings.

(13) It is often necessary to estimate the maximum depth of slide — from the ground surface to the surface of rupture—and it is of great significance as a guide to determining the magnitude of the slide and to the depth to which subsurface exploration will have to be carried out. Adopt a quick method which permits a fairly reliable estimate for preliminary purposes which is based on a minimum of required observational data, such as methods like (a) slip circle method, and (b) concentric circle method, discussed in Chapter 5.

(14) Tables 8 summarises the surface features of the various parts of active or recently active slides as they aid in the identification of different landslide types. On the basis of data already collected and using the above table as a guide, identify the type of slide that you are dealing with.

(If a landslide develops as a slump and over a period of time turns into a flow, the original report on the nature of the slide would not be valid as a basis for planning the correction of the slide at a later date. Therefore, the identification of the type of slide should be made at the same time as it is to be corrected. Even when a landslide that starts as a slump but later changes into a flow, is corrected as a flow, this does not necessarily mean that the adjoining area which may still be a slump block, or yet another area that has not moved at all, should require the same kind of correction. Each slide should be classified according to its own characteristics at the time it is to be corrected. If this not done, time may destroy the value of identification work and a corrective procedure based on the previous characteristics of the slide is likely to be a wrong one).

(15) For a description of the salient features of falls, slides and flows, please see Chapter I.

(16) Once the fact of land movement has been established, the next essential step is to identify the type of landslide as suggested in rule No. 14 above. Having identified the landslide type together with the necessary data (collected in the shape of answers to the Questionnaire appended to the Handbook), make an appropriate choice of the corrective measures on the basis of reasoning given in Chapter 6.

(17) For purposes of slope design in soil cuts and in bed rock cuts, please see Chapter 2 and Chapter 3 respectively.

(18) If the types of soil dealt with is essentially detritus, loose sand, soft homogeneous clay, stiff fissured clay or loess, please see Chapter 4 and Chapter 6 for selecting the appropriate corrective measures.

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM
SYNTHESIS OF HIGHWAY PRACTICE **8**

CONSTRUCTION OF EMBANKMENTS

RESEARCH SPONSORED BY THE AMERICAN ASSOCIATION
OF STATE HIGHWAY OFFICIALS IN COOPERATION
WITH THE FEDERAL HIGHWAY ADMINISTRATION

AREAS OF INTEREST:
- HIGHWAY DESIGN
- CONSTRUCTION
- EXPLORATION-CLASSIFICATION (SOILS)
- FOUNDATIONS (SOILS)
- MECHANICS (EARTH MASS)

HIGHWAY RESEARCH BOARD
DIVISION OF ENGINEERING NATIONAL RESEARCH COUNCIL
NATIONAL ACADEMY OF SCIENCES—NATIONAL ACADEMY OF ENGINEERING 1971

CONTENTS

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CONSTRUCTION OF EMBANKMENTS

SUMMARY

Highway embankment failures may result from a variety of causes. The satisfactory performance of an embankment is dependent on its own stability and that of the underlying foundation. Currently, more embankment problems are the result of poor foundations than of faulty placement of the embankment itself. This points out the need for an adequate subsurface investigation of the foundation conditions at embankment sites.

The route location phase should include provisions for considering sites where subsurface conditions could cause embankment problems. Examples of these sites include areas of potential landslides, sidehill cuts and fills, abandoned mining operations, soft foundations, buried stream channels, and sanitary landfills. Soil surveys provide detailed information of specific conditions along the selected route. In many agencies, there is a need to increase the depth of exploration in embankment areas to sample the materials affected significantly by embankment loads.

Embankment construction begins with the preparation of the foundation. Any unusual condition likely to cause problems during construction or during the life of the embankment should have been identified and corrective measures prescribed prior to start of construction. Consolidation of soft foundation material is a widespread problem. The difficulties associated with sidehill fills and cut-to-fill transitions are also classified as critical by many agencies. Benching and adequate drainage provisions are the most widely accepted solutions for correcting both sidehill fills and cut-to-fill transitions.

In the design of embankments, geometric design criteria and safety standards usually have precedence over other considerations such as soft foundation and poor materials. The design of high embankments should consider the quality of the fill materials because the weight of the embankment is critical.

Standard specifications of highway agencies continue to require fill material to be placed in relatively thin lifts and compacted by rolling with suitable equipment. There is, however, a trend toward minimizing procedural specifications and placing greater reliance on density requirements. Moisture content is a continuing problem, particularly with silty soils and swelling clays. Current procedures for building rock embankments are generally satisfactory. A move to thicker lifts may be justified with vibratory compactors.

Sand-cone and balloon methods are the two most prevalent test methods used to determine in-place density. Both are time consuming. The use of nuclear equipment to perform nondestructive density and moisture measurements is increasing. In-place densities are most commonly evaluated in relation to the AASHO T-99 maximum density. The most common procedures for the field evaluation of maximum dry density involve the use of a one-point compaction test with a family of moisture-density curves.

Density requirements, except specifications based on statistical quality control, are considered to be minimum standards that must be exceeded by all field test results. Statistical concepts for density requirements help to evaluate the significance of an occasional bad test.

Problems with expansive clays are more common in cut sections than in

embankments. It is usually possible to use these soils in the fill so that they do not represent a major problem to the completed construction. However, special methods and controls may be required.

Frozen soils cannot be compacted satisfactorily. Unless the frozen layer of material can be removed from both the embankment and the cut or borrow area, operations should be suspended.

Although construction practices directed toward protecting the environment have always been encouraged, improved practices are being introduced that will have a significant impact on highway construction. Waste disposal and erosion control are areas that are receiving much attention.

For successful earthwork construction, it is important to integrate design, construction, and soils considerations, beginning with the initial route location studies and continuing through the completion of construction.

CHAPTER ONE

INTRODUCTION

Geometric design and other route location considerations often require modern highways to be constructed at elevations above existing ground. Although such conditions have long been faced at water crossings and in mountainous or hilly terrain, the limited access and more stringent geometric design requirements of the Interstate Highway System have introduced the problem of supporting pavement systems above ground in areas of flat topography. When a pavement is to be supported above existing ground, a bridge structure or an earth embankment is required. At water crossings, narrow ravines, and some sites of extremely poor foundation conditions, bridging usually will be the most satisfactory solution. In some other cases, strong consideration should be given to adjustment of the center line or the grade so as to place the pavement system at or below natural ground. However, in the vast majority of situations, an earth embankment will provide the most practical and economical road support system. As a consequence, embankment construction is a major component of modern highway construction.

EMBANKMENT FAILURES

The function of a highway embankment is to provide support for a pavement system above natural ground. An embankment has failed when it causes roughness or damage to the roadway. The failure may be spectacular and catastrophic, as in the case of slides resulting from instability of the embankment or the underlying foundation materials. A total loss of the pavement section and portions of the embankment itself may result (Fig. 1). However, the failure is usually more subtle. Creep and/or consolidation of the embankment or the underlying foundation materials may produce failure by the gradual development of excessive differential settlements of the pavement surface, causing rutting, dips, or cracks (Fig. 2). Thus, embankment performance is associated with the stability and the deformation of both the embankment and the underlying foundation materials.

There are many reasons for unstable embankments. Failures within the foundation may be the result of inadequate site investigations, insufficient consideration of foundation conditions in design, or improper implementation of the design solutions during construction. Failures originating within the embankment itself may be caused by poor materials, unsatisfactory construction methods, or ineffective quality control procedures. Currently, more embankment problems are the result of poor foundation conditions than of faulty placement of the embankment itself. These foundation problems are caused primarily by inadequate consideration of soft foundation soils, sidehill locations, cut-fill transitions, and groundwater conditions. Relatively few problems result from poor placement of fills, because generally good construction practices and quality control procedures for placement of fills have been developed throughout the past 30 years.

EMBANKMENT LOAD

Highway embankments are major structures that produce significant loads on the underlying foundation soils. The weight of each foot of fill is roughly equivalent to the weight of one story of a conventional office or apartment building. In other words, a 20-ft-high earth embankment

will weigh as much as a 20-story office building occupying an equivalent contact area. Furthermore, the rate at which the contact pressure is dissipated with depth beneath the natural ground surface depends on the least dimension of the loaded area. Because of the great width of many embankments, the stress beneath the center of the embankment usually decreases very slowly with depth. The combination of these factors means that large stress increases can be produced at significant depths beneath an earth embankment.

The preceding principles are illustrated by the example shown in Figure 3, in which the stresses produced by a 20-ft-high earth embankment are compared to those produced by a bridge pier foundation. Although the contact pressures are much higher for the bridge foundation, the stresses decrease very rapidly with depth. At depths of more than 10 ft, the stresses caused by the embankment loading exceed those of the bridge foundation. At a depth of 80 ft, the stresses caused by the embankment loading are still more than 80 percent of the surface contact pres-

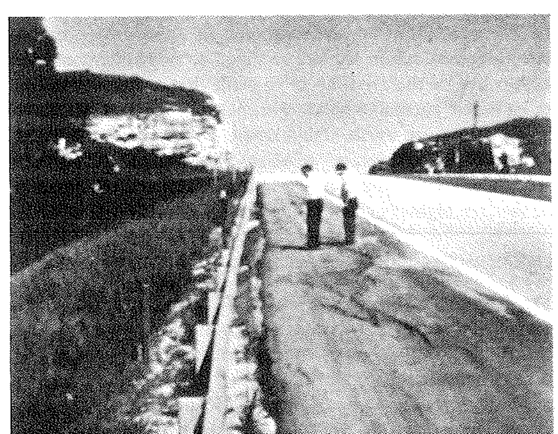


Figure 2. Visual indication of embankment failure.

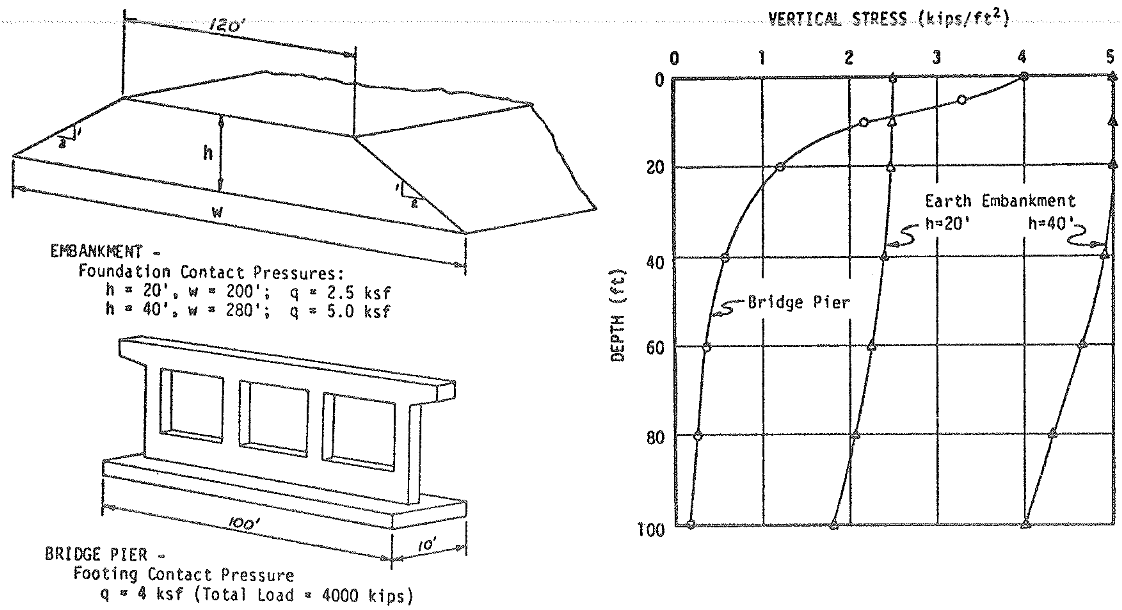


Figure 3. Comparison of vertical stresses beneath center lines of bridge pier and earth embankments.

sure and are more than eight times greater than those under the bridge foundation. The differences are even more dramatic for the stresses beneath a 40-ft-high embankment, also plotted in Figure 3. Hence, an adequate subsurface investigation of the foundation conditions beneath embankment sites becomes an essential prerequisite for satisfactory embankment design and construction.

SCOPE OF SYNTHESIS

For earthwork construction to be successful, materials, design and construction considerations must be intermeshed from the initiation of the site investigation through the completion of construction. Consequently, construction practices cannot properly be separated from design and materials considerations. For this reason, the synthesis includes:

1. Subsurface investigational practices for embankment areas.
2. Design and construction procedures for the treatment of embankment foundations.
3. Design criteria, specifications, and construction practices for embankments.
4. Quality control procedures for placement of embankment materials.

Both good practices for normal conditions and special practices for handling unusual conditions are presented.

Cut zones are not considered except as they affect embankment construction (e.g., as sources of embankment materials or special procedures at cut-fill transitions). Special treatments or procedures applied to the upper surface of an embankment are regarded as part of the pavement construction and have been excluded. Furthermore, the special problems associated with the design and construction of bridge approaches have been presented in *NCHRP Synthesis 2*, so are not discussed in detail herein.

CHAPTER TWO

SUBSURFACE INVESTIGATIONS

Adequate knowledge of subsurface conditions often is the key to successful earthwork construction. Most agencies have long recognized the need for comprehensive subsurface investigations at bridge sites. Currently, there is a growing awareness of the potential effects of the extremely heavy loads imposed on the subsoils by embankments and, as a consequence, the investigational programs for embankment foundations are being upgraded.

Subsurface information can be used at several stages in the development of a highway project. First, subsurface conditions can be an important and sometimes decisive factor in route location. Second, after specific lines and grades are established, detailed subsurface investigations are required for the design of embankment sections.

ROUTE LOCATION

Subsurface conditions can significantly affect the cost of highway construction and hence should be one of the considerations in highway location studies. Although other factors often will outweigh the influence of the subsurface conditions, many examples can be cited of locations where the subsurface conditions were or should have been the decisive factor. These examples include locations in areas of potential landslides, sidehill cuts and fills, abandoned mining operations, limestone cavities, and various soft foundation conditions, such as peat bogs, marshes, buried stream channels, and sanitary landfills.

The expertise of the geologists and soil engineers who are responsible for the detailed soil surveys, conducted after the location has been established, should be used in the preliminary investigations. When the preliminary investigation is conducted by special reconnaissance personnel, trained to consider a variety of route location factors, it is essential that these personnel also have sufficient training and experience to recognize potential geotechnical problem areas for which the advice of the specialists should be sought.

Several agencies have successfully instituted a policy of preparing preliminary geotechnical reports for major route location studies. These reports are prepared by geotechnical specialists in advance of public hearings so that subsurface conditions can be considered fully together with other route location factors. Preliminary geotechnical reports generally are prepared from existing information and from limited field reconnaissance. Sources of information include published and unpublished geological surveys and maps, groundwater and hydrologic surveys, topographic maps, soil conservation reports, agricultural soil maps, air photos, and existing detailed soil surveys from previous highway projects in the vicinity of the proposed route. Field reconnaissance commonly is limited to recollections of past personal experience or to walking the line of each proposed route. Occasionally, geophysical exploration techniques, such as seismic and resistivity surveys, are used; and in a very few instances, when extremely critical conditions are identified, probe borings and/or samples may be justified. The preliminary geotechnical report includes general descriptions of the topography, the geology, and the soil conditions along each proposed alignment. Typical profiles or geologic cross sections are desirable. This information is used to identify potential problem areas, including consideration of the potential for landslides, stability of cut and fill slopes, extent of rock excavation, groundwater conditions, potential foundation conditions for bridge structures and embankments, and availability and quality of construction materials. The potential need for special foundation treatments may be noted.

The potential of air photos for the preparation of preliminary geotechnical reports deserves special consideration. Almost all highway agencies currently are obtaining air photos of proposed route locations for photogrammetric purposes; i.e., preparation of topographic maps, right-of-way acquisition, and earthwork calculations. These same air photos also can provide qualitative information on the soil and rock conditions.

SOIL SURVEYS

After the line and the grade for a proposed route have been established, a detailed soil survey is made. It includes a field exploration program, laboratory testing, and, usually, some soil mechanics analyses. The report of the soil survey generally includes detailed descriptions of the soil and rock materials encountered along the right-of-way, a plot of the soil profile, and a summary of all laboratory test results. Problem areas are identified and design solutions proposed. Also, the quality and quantity of materials sources for embankments, subgrades, and pavement components are reported. Several agencies (e.g., Kentucky) have prepared extensive specifications for the preparation of a soil survey. These manuals are excellent sources of detailed information on the requirements of a satisfactory soil survey.

The findings and recommendations of the soil survey must be adequately reported to the engineers responsible for both the design and the construction of the embankment. It is highly desirable for the boring logs and soil profiles to be plotted on the construction plans.

Field Exploration Procedures

Typically, both geophysical and soil boring techniques are employed for the soil survey. The most common geophysical methods are seismic refraction and electrical resistivity. These techniques are most useful in determining the position of the groundwater table, the depth to rock, and the delineation of various rock strata. They are less effective in differentiating among soil types and do not provide for visual identification of the subsurface materials. Thus, when these procedures are used, they are almost always supplemented by some sort of boring program.

Current good practice for embankment foundation investigations includes the use of disturbed borings and samples supplemented with undisturbed sampling in critical areas. The use of a hand or power auger to obtain disturbed samples is by far the most common soil exploration technique. In addition, split-barrel samplers (standard penetration tests), which provide undisturbed samples, or static cone penetrometers sometimes also are used. When soft deposits of cohesive soils are encountered in embankment areas, undisturbed samples commonly are obtained with hydraulically operated thin-walled samplers. Most borings for embankment foundations are terminated when rock is encountered. In some instances, rotary or diamond core borings are used to investigate the soundness of bedrock or to obtain samples of soft weathered rock. For economic reasons, undisturbed sampling techniques generally are limited to critical problem areas that previously have been identified from disturbed borings.

The spacing between borings will depend on the specific geologic conditions along the right-of-way and the need for exact data by the designer for specific locations on the highway. Because the critical areas commonly represent only a small percentage of the total investigation, heavy reliance must be placed on the judgment and experience of field personnel to obtain adequate borings in all potential problem areas. At least one boring should be made beneath each fill. For long embankments the maximum interval between borings can range from 250 to 1,000 ft, with spacings of 300 to 500 ft most common. In problem areas the spacing between borings will be significantly reduced and may approach 50 ft when defining the extent of soft deposits. Typically, borings are located along the center line of the pavement, except when defining the limits of soft deposits or investigating sidehill locations.

Many agencies continue to specify inadequate minimum depths for borings in embankment areas. The required depth of borings should be related to the width and height of the proposed embankment. On the basis of the depth to which significant stresses are produced by embankment loads, borings ought to extend to depths equal to the half-width (i.e., the distance from the center line to the toe of the slope) or to twice the height of the embankment. The width criterion is fundamentally more correct, but the height criterion is easier to use in practice. These criteria may be tempered by experience with local geologic conditions. Also, borings may be terminated at shallower depths when firm bedrock is encountered. Conversely, borings never should be terminated in a soft deposit, but should continue until firm material is located.

Laboratory Investigations

Routine laboratory investigations include mechanical analyses and Atterberg Limits tests for classification of all soils encountered along the right-of-way. Depending on the soil classification system used within a specific agency, additional tests, such as volume change, swell potential, sand equivalent, resilience, or compaction tests, may be required of all samples. In addition, field densities and moisture contents are determined. Procedures for conducting all tests are detailed in each agency's test manual or by reference to AASHO or ASTM standards.

When the strength or the compressibility of the foundation soil is to be determined, undisturbed thin-wall samples are essential. Relatively conventional consolidation tests are required when soft compressible foundation materials are identified and settlement analyses are to be made. If the stability of a soft foundation or a sidehill is of concern, the shear strengths of the subsoils are determined from laboratory unconfined compression, direct shear, or triaxial compression tests. The type of test may be related to the method of stability analysis and the factors of safety that will be employed. Carefully performed simple tests can be more meaningful than poorly conducted sophisticated tests.

EMBANKMENT SOILS

The field investigation of soils to be used in embankments generally is limited to auger borings. The laboratory testing of these soils includes the gradation, plasticity, and miscellaneous tests required for classification. Some type of compaction test is performed to determine a standard maximum density and the optimum moisture content at which it is obtained. Frequently, a stiffness test, such as a CBR or stabilometer test, is conducted on the compacted material for use as a subgrade property in pavement design. Shrink-swell factors may also be estimated for use in earthwork computations.

Currently, there is only an occasional need for strength and consolidation tests of embankment soils. Laboratory and field data indicate that the compression of an embankment that has been compacted to current density requirements is usually negligible. Similarly, for the current standard design slopes, the strength of an embankment generally is not critical. Exceptions may occur in embankments exceeding 50 ft in height or when extremely poor quality soils must be used. In these instances, strength and consolidation tests should be conducted on the compacted embankment soils. Although there is minimal need for these tests today, it is likely that their use will become more common in the future as higher embankments are constructed and lower quality soils must be used more frequently.

CHAPTER THREE

FOUNDATION PREPARATION

The construction of an embankment begins with the preparation of the foundation. When good foundation conditions are encountered, only simple straightforward procedures are required. However, when poor foundation conditions are involved, complex and expensive treatments may become necessary to prevent poor performance or embankment failure. Soft foundation soils, sidehill locations, cut-fill transitions, and groundwater problems are examples of conditions that generally require special treatment. The occurrence of these four conditions is sufficiently widespread that their treatments are discussed in detail.

Because special foundation treatments can be very expensive, it is extremely important that the need for special treatments be identified in the soil survey and adequately considered in the preparation of design drawings and specifications. When unanticipated problem areas are encountered during construction, corrective measures not only delay construction but also inevitably cost more than if anticipated in the design. Thus, planning for treatment of poor foundation conditions becomes an important aspect of embankment design. Recognition and analysis of poor foundation conditions and recommendations for corrective treatments generally are the responsibility of the geologists, and soils or materials engineers who prepare the soil survey.

MINIMAL PREPARATION

Clearing and grubbing are the first steps in the preparation of an embankment site. The topsoil usually is removed. In many instances, all stumps also are removed from the right-of-way. However, because of increasing costs of removal and problems of disposal, it is becoming increasingly common to allow stumps to remain in place when they are a sufficient distance below the subgrade. Typically, trees may be cut within 3 or 4 in. of natural ground and the stumps left in place when the embankment height is greater than 5 or 6 ft.

For very low embankments, the foundation material may be undercut to improve the uniformity within the subgrade zone and to eliminate some cut-fill transitions. Generally, the depth of undercutting is specified so as to produce a minimum embankment height of approximately 3 ft. At least one agency is undercutting most foundation materials to the ditch line and constructing low embankments, even in cut zones. More commonly, the natural ground below shallow fills is proof rolled or compacted in place. If soft spots are encountered, undercutting may be required. In some instances the excavated material can be dried and then recompacted; in others the excavated material must be wasted and replaced by acceptable material.

If satisfactory foundation conditions are encountered, the preceding procedures are all that are normally required and the construction of the embankment itself can be started. If unanticipated foundation problems are exposed during these preliminary operations, geologists or soil specialists should be consulted for recommendations of additional treatments. If the poor conditions were anticipated in the design, additional corrective treatments already will have been prescribed in the plans or specifications.

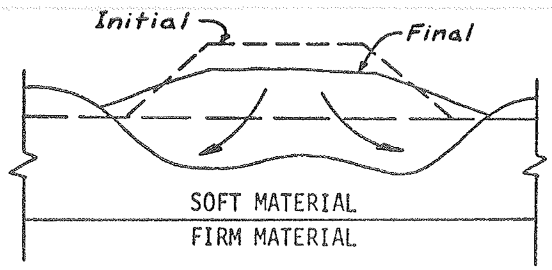
SOFT FOUNDATIONS

Soft foundation soils include a variety of peats, marls, and organic and inorganic silts and clays. These deposits may be localized, as in a peat bog or a river crossing, or they may encompass vast areas, such as tidal marshes or glacial lake beds. The compressible material may occur at the surface (e.g., marshes and some peat bogs), or it may be buried beneath a mantle of satisfactory soil. Examples of the latter case include abandoned stream channels, lake beds, and peat bogs, which may be covered by a dessicated crust or a more recent soil deposit.

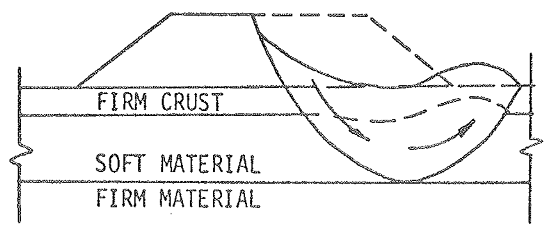
When soft soils are discovered, both the strength and the compressibility testing, and analysis, will depend on the magnitude of the project and also the types of treatment being considered. For example, if it is planned to excavate the soft material, extensive knowledge of its properties usually is not required. On the other hand, if a sand drain installation is contemplated, a very extensive investigation is necessary.

Soft foundations create several types of embankment problems. The foundation soil may displace laterally under the weight of the fill, causing major movements and disruptions of the embankment, as shown in Figures 4a and 4b. If the lateral displacement is prevented, the foundation soil still will compress and cause settlement of the embankment and the pavement, as shown in Figure 4c. Some settlement is to be anticipated in almost all embankment foundations; it is detrimental only if it produces fracture or excessive roughness of the pavement.

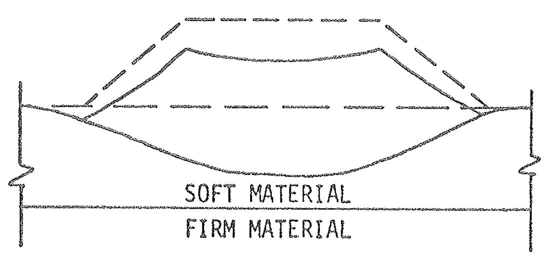
The total or differential settlement that can be tolerated by a pavement rarely is specified except in the case of bridge approaches, for which the tolerable settlement commonly is specified as ½ to 1 in. For roadway embankments, the allowable settlement after paving depends on the length of the fill and the rate at which settlements develop. Experience has indicated that 6 to 12 in. of settlement can be tolerated in long embankments, if any variations in the settlement are uniformly distributed along the length of the embankment. At least one agency permits a differential settlement of 2 in. per 100 ft. In some instances, predicted settlements of several feet can be accepted if the



a) Embankment Failure Over Soft Foundation



b) Embankment Failure - Soft Material Overlain by Firm Crust



c) Settlement of Embankment Over Soft Foundation

Figure 4. Typical embankment problems over soft foundations.

settlement develops very slowly over long periods of time (e.g., 25 to 50 years). In general, however, the amount of settlement that can be tolerated by a pavement system is not well defined and requires further study.

When long-term settlement is anticipated, flexible pavements often are used to facilitate future maintenance. Also, interim or stage construction of the pavement sometimes is employed. These procedures may be more economical than attempting to improve the soft foundation conditions prior to construction of the pavements. In some critical situations, these procedures may be necessary in addition to some initial partial treatment of poor foundation conditions.

Several treatments commonly are used to improve soft foundations. They include (a) removal by excavation or displacement and (b) consolidation by preloading with a waiting period or surcharging with or without sand drains and berms. Another procedure is the use of lightweight embankment materials, perhaps in combination with one of the preceding treatments.

Removal

Removal by excavation is perhaps the most common method for treating soft foundation conditions. Excavation usually is the most economical and satisfactory solution when the unsuitable material does not extend to depths of more than 15 or 20 ft. Beyond this depth, alternate solutions may be more effective than excavation. Of course, the depth beyond which excavation ceases to be the most economical treatment for a specific site will depend on the soil type and groundwater conditions at the site. Examples can be cited in which as much as 40 ft of soft material have been successfully excavated.

When excavation is employed, the depth of excavation should extend to firm soil. However, the required lateral extent of excavation is less clearly defined. Four typical methods for defining the zone of excavation are shown in Figure 5. In examples (a) and (b) the width of the zone of excavation depends only on the height and slope of the embankment, whereas in examples (c) and (d) it also depends on thickness of the unsuitable material. The use of the section in Figure 5 is restricted to deposits that are less than 5 ft thick. Wide zones of excavation reduce the danger of lateral creep of the embankment.

The chief advantage of complete removal by excavation is the certainty that the potential foundation problems will be eliminated. The method is most effective for relatively shallow surface deposits of limited extent (for example, peat bogs and marshes). It becomes costly and impractical for extensive deposits of compressible clays or deposits that are buried beneath a mantle of suitable material. Also, every cubic yard of excavated material must be replaced by an equivalent quantity of good fill. Thus, the availability of sufficient quantities of suitable fill material becomes one criterion for the effective use of the excavation method. Furthermore, if unsuitable material is excavated below the groundwater table, special placement procedures may be required to dewater the excavation or to place the fill under water.

Removal by displacement is an old technique that has been used effectively on peat and marsh deposits. Today, however, displacement methods are used infrequently and with much caution because of the uncertainty of complete removal. Displacement is accomplished by controlling the position of fill placement so as to displace the unsuitable material laterally and ahead of the embankment construction. Usually the displacement force is produced by the weight of fill, but occasionally it is supplemented by blasting, jetting, or surcharge. The filling operation must be controlled very carefully to prevent entrapment of pockets of unsuitable material. Because of these uncertainties, detailed specifications and careful field inspection are necessary when displacement procedures are employed. Core samples of the completed embankment are desirable to verify that no pockets of unsuitable material remain.

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Consolidation

Other treatments involve the consolidation of the soft foundation materials prior to finished grading and construction of the pavement section. Because consolidation is a time-dependent process, these techniques generally require a delay or waiting period between completion of the embankment and construction of the pavement section.

When the consolidation of soft materials is considered, it is necessary to estimate both the magnitude and the time required for development of the ultimate consolidation of the soft foundation under the loads imposed by the embankment and pavement system. Also, the stability should be investigated to ensure that the stresses produced by the embankment loads do not exceed the strength of the subsoil and produce massive slides and uncontrolled lateral displacement of the foundation soils. These analyses, together with the related consolidation and strength tests of the foundation soils, must be included in the soil survey whenever consolidation procedures are recommended.

If the predicted settlement is more than can be tolerated by the roadway, first consideration is given to constructing the embankment to subgrade elevation and waiting for completion of the consolidation due to the embankment loading prior to final grading and construction of the pavement section. In this case, the predicted magnitude of settlement is used only to estimate the additional quantities of fill required and thus is of secondary importance. However, when settlements of several feet are produced, the additional fill required to compensate for the settlement can become a significant percentage of the total earthwork. The primary consideration when delaying pavement construction is the time required for completion of the consolidation process. One year usually is the maximum acceptable delay period. Occasionally, when soft foundation problems are recognized far enough in advance, scheduling can be arranged to allow as long as two years for consolidation. More commonly, however, when the predicted delay periods are longer than a year alternate methods are considered to accelerate the rate of settlement. Because the primary consolidation time is related to the square of the drainage distance within the compressible soil, the necessity of additional treatments becomes increasingly probable with increasing thickness of the soft layer.

The most common procedure for accelerating the rate of settlement is application of a surcharge load, which is produced by constructing the embankment to a height in excess of the design height. Settlements will occur more rapidly under the combination of the design and the surcharge loads than under the design load only; hence, the waiting period will be decreased. Sufficient fill must be available to develop the surcharge. Also, the addition of a surcharge load increases the prospects for stability problems and may require additional adjustments in the embankment design (for example, flattening of slopes or construction of berms) to compensate for this.

The rate of settlement also can be accelerated by improving the drainage of the compressible layer with sand blankets or vertical sand drains. Placement of a thick granular layer over the compressible stratum prior to constructing the embankment insures free drainage of the upper surface of the compressible layer. It does not, however, appreciably shorten the drainage path in the compressible layer; hence, its effectiveness is limited to relatively thin surface deposits of compressible material. On the other hand, installation of vertical sand columns through the compressible stratum can appreciably reduce the drainage path by developing radial flow to the pervious

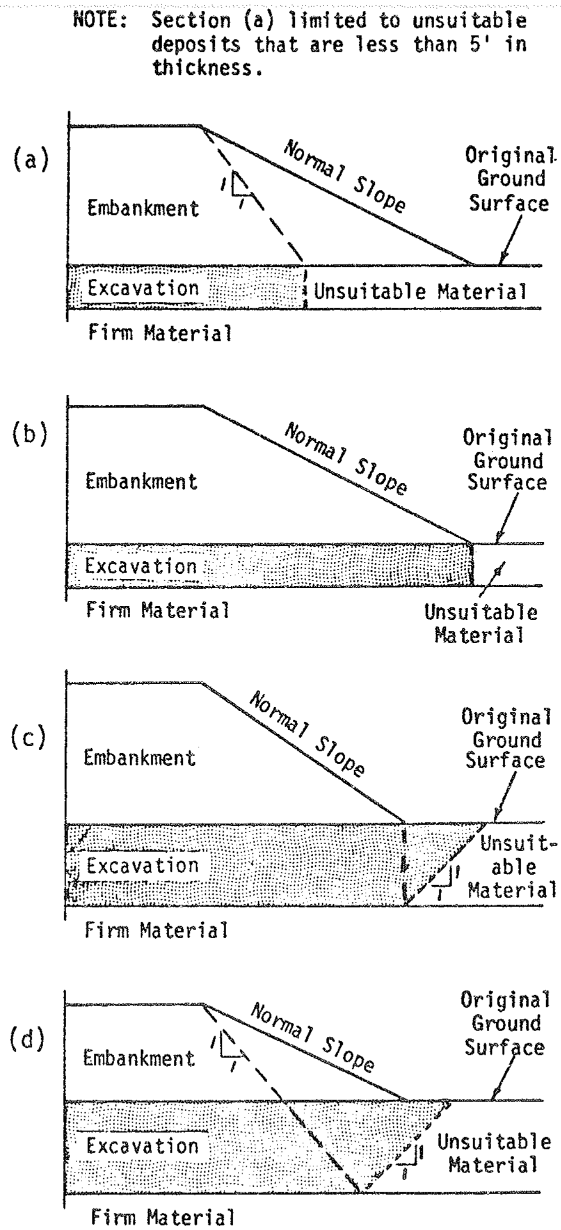


Figure 5. Examples of excavation of unsuitable material.

sand columns. The radial drainage distance, hence the time required for consolidation, are controlled by the spacing between drain wells. Thus, the primary consolidation time can be reduced from many years to several months by the installation of a properly designed vertical sand drain system. Surcharges commonly are employed in conjunction with sand drains and the benefits of both techniques are combined. Surcharging also will decrease the amount of post-construction secondary compression.

Theories and design procedures for sand drain systems now are readily available in the soil mechanics literature. Moreover, because the initial installation by an agency often has been treated as a research project, an increasing number of well-documented case studies of instrumented sand drain installations are becoming available. Although many good case studies remain unpublished, excellent reviews of the design and installation of sand drains are found in Moore and Grosert (1968), Johnson (1970), and Moran, Proctor, Mueser and Rutledge (1958).

As effective as the sand drain method may appear, it is used infrequently in highway construction. The foremost reason is that sand drain installations are relatively expensive and should be considered only when other methods are not adequate. Also, sand drains may not be suitable for certain very soft deposits. For example, in very soft marsh deposits, lateral movements under the embankment may tend to disrupt the vertical sand column and eliminate its effectiveness.

Considerable controversy remains in regard to various methods for installing sand drains. The driven closed-end mandrel procedure is most common because it is rapid, efficient, and the least expensive installation method. However, it is also the most criticized method because large volumes of soil are displaced and the resulting soil disturbance may reduce stability and increase compressibility.

Nondisplacement installation methods, such as jetted mandrel or hollow-stem auger methods, are believed to reduce soil disturbance effects but are more expensive than driven mandrel installations. Some agencies now ban the use of the driven closed mandrel and require more expensive nondisplacement installations; others believe that the advantages of nondisplacement methods have not been sufficiently documented to justify their higher cost. Few field studies of the relative effects of various installation methods are available. Two excellent unpublished field studies in New Hampshire and Maine have led to the recommendation of jetted nondisplacement methods for sand drain installations in sensitive soils in these states. The installation method was less significant in relatively insensitive soils.

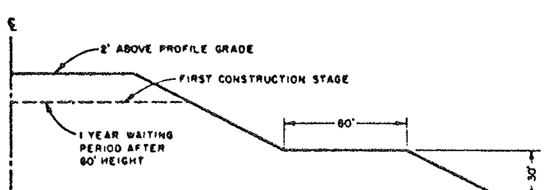


Figure 6. Typical embankment with berms and stage construction.

In all preloading methods, with or without surcharges and/or sand drains, the stability of the embankment and foundation system must be considered. The danger of lateral instability generally is increased by the use of a surcharge and sometimes by the installation of sand drains. A factor of safety against sliding of at least 1.5 usually is required. If lower factors of safety are calculated, the embankment section may be redesigned by either flattening the slopes or adding berms. The choice between these two methods is based on consideration of right-of-way limitations and ease of construction and maintenance When berms are used, it is important that they be sufficiently wide to be effective. This must be evaluated by additional stability analyses of the revised embankment section. Frequently, the berm must extend laterally to intersect the original critical stability circle. A typical embankment section with berm is shown in Figure 6.

In a relatively few instances, stage construction of the embankment is used to overcome stability problems. Because the foundation soils become stronger as they consolidate with time, the factor of safety against sliding is most critical initially. The rate of construction is controlled so as to take advantage of the increases in strength with consolidation. Construction rates may be controlled by specifying an incremental construction height to be followed by a waiting period with no construction prior to starting the next incremental height, or by specifying a continual maximum rate of construction (e.g., 5 ft per week). Stage construction should be avoided whenever possible, because of the close construction supervision that is required for it to be successful.

In all instances where soft foundation materials are being consolidated, field instrumentation and close construction supervision are imperative. Because of the many uncertainties involved in predicting field settlement rates from laboratory tests, field measurements commonly are used as guides to control the rate of construction or the duration of waiting periods. The extent of the field instrumentation will depend on the complexity of the project. On simple projects, only measurement of the settlement of the embankment surface may be necessary. However, often it is also desirable to measure the settlement at points within or at the base of the embankment. As the projects become more complex and stability becomes a major consideration, the pore pressures within the soft material and the lateral movements of the embankment and the foundation also may be measured. These measurements are needed particularly in projects involving sand drains, berms, or stage construction. A discussion of the types of instrumentation is presented in Chapter Seven.

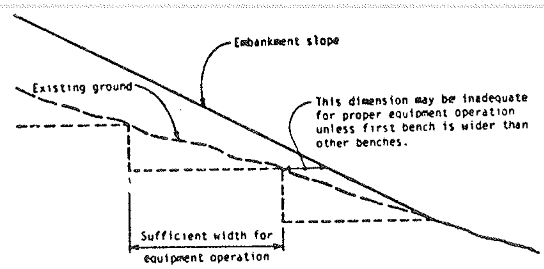
Stability and settlement problems also can be reduced through the use of lightweight embankment materials. However, applications of this technique are extremely rare, primarily because of the limited availability or the prohibitive cost of lightweight aggregates. Also, experiments with various methods for deep lime stabilization of soft soils have been ineffective or prohibitively expensive.

SIDEHILL FILLS

Sidehill fills are cited by many agencies as their most critical embankment problem. Instability and sliding generally are associated with two geologic factors common to sidehill locations. First, in many locations bedding planes or boundaries between weathered and unweathered material tend to slope downhill. The addition of a sidehill fill often produces a tendency for sliding along these natural planes of weakness. Second, the construction of a sidehill fill disrupts the natural movements of surface water and groundwater. The accumulation of water within the embankment zone increases the tendency for sliding by increasing the weight of the sliding mass and at the same time decreasing the soil's resistance to sliding. Even when water does not appear on the surface, the accumulation of groundwater along the sloping bedding planes within the hill can produce a slippery plane along which sliding is likely to develop. Thus, groundwater control becomes a major consideration in the construction of satisfactory sidehill fills. However, before the groundwater can be controlled, its presence must be identified. This is often difficult during field surveys because the groundwater conditions are likely to fluctuate throughout the year, and the most critical conditions may not exist when the field survey is made.

Because of the difficulties in identifying and correcting sidehill problems, strong consideration should be given to adjusting alignments so as to avoid hills. However, adjustment often will not be possible and the sidehill site must be used. In these cases, special construction practices will consist of some type of benching to key the embankment to a firm foundation and special drainage provisions to prevent the accumulation of surface water and groundwater.

Benching consists of excavating into the sidehill to establish a horizontal platform upon which to construct the embankment. Benching is usually required when the natural slope exceeds 4:1 or 6:1. The specific dimensions of the benches may be given on the design drawings or in special provisions for each project. Sometimes the actual sizes of the benches on specific jobs are established in the field during construction. Often the minimum width of a bench is established on the basis of the width of construction equipment. There is a tendency to make benches too narrow. To be effective, they must be wide enough to allow the embankment to be anchored in firm material. This means, for example, that benches should intercept the transition zone between weathered soil and the unweathered parent material. Typical longitudinal benches are shown in Figure 7.



NOTE: Benching is usually required if slope of existing ground is greater than 4:1

Figure 7. Cross section of typical longitudinal benches.

A variety of drainage control procedures can be used for different groundwater conditions. Many of these, not particularly limited to sidehill locations, are discussed in the section on "Drainage Provisions."

Stabilization trenches are used on some sidehill locations. Although the trenches are constructed primarily to control deep groundwater, they also key the embankment into stable underlying foundation soils. Stabilization trenches are excavated with a bottom width of at least 12 ft and side slopes as steep as possible. The depth of the trench is determined by the position of the groundwater to be intercepted. Excavations have ranged as deep as 35 ft below center line. The bottom, high side, and ends of the trenches are blanketed with a 3-ft layer of permeable material and perforated pipe is laid in the bottom of the trench. Short transverse trenches with perforated pipe are constructed at intervals along the main trench to provide a drainage outlet. The embankment is then started by backfilling the stabilization trench. A schematic diagram of a stabilization trench is shown in Figure 8.

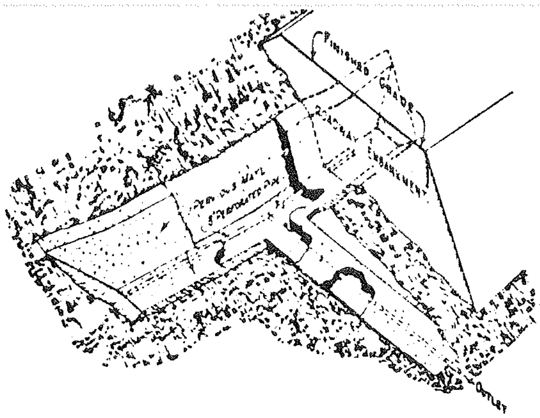
CUT-FILL TRANSITIONS

Cut-fill transitions basically are transverse sidehill locations and thus involve the same considerations discussed in the preceding section. Benches and drainage systems may be required. However, in order to maintain a uniform subgrade, some details of the benches at cut-fill transitions differ from the bench sections in sidehills. The bench must extend sufficiently into the cut zone to ensure that all unstable soil is removed from the subgrade zone. In addition, the uppermost bench is tapered to provide a gradual transition from the embankment to the natural ground. An example of typical benching practice at a cut-fill transition is shown in Figure 9.

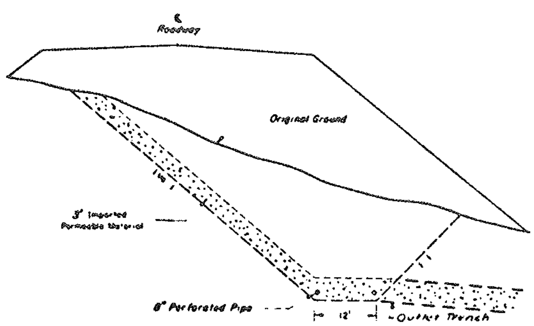
Drainage is particularly critical at transitions because the cut slopes provide a potential source of groundwater. As a result, it is good practice to provide for drainage at all cut-fill transitions. A simple pipe installation is shown in the benched section in Figure 9. If necessary, transverse stabilization trenches or other more complex systems can be constructed.

DRAINAGE PROVISIONS

The control of groundwater is an important aspect of embankment construction except in arid regions. A variety of subsurface drainage systems are used in current practice. Except in a few special cases, they consist of pervious



a. Diagrammetric sketch



b. Cross section

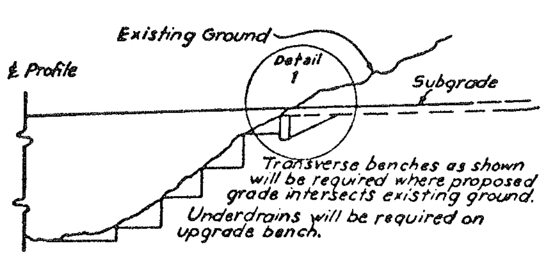
Figure 8. Longitudinal stabilization trench.

blankets and/or some type of drain pipe system. Most of these subdrain systems are very effective when they are designed with sufficient capacity and placed in the proper location.

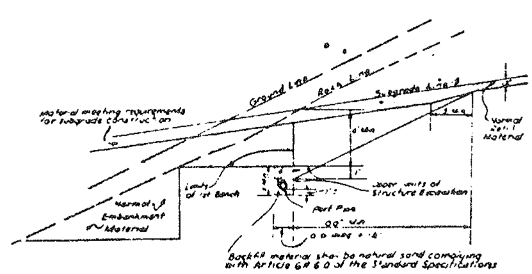
Drainage provisions for groundwater control can be very expensive. For example, one agency has indicated that the cost of subsurface drainage facilities represented approximately 10 percent of one $5 million project and almost 14 percent of another $11 million project. Thus, the economics of groundwater control facilities becomes an important consideration. It may be extremely uneconomical to provide subdrain facilities in locations for which their need is not indicated by the soil survey. On the other hand, the consequences of omitting drainage provisions where they actually are needed can be even more expensive. To quote one soils engineer: "To ignore the subsurface water in the construction of cuts or fills . . . in the hopes that construction or natural conditions will improve the subsurface drainage, can be a disastrous and costly process. Almost without exception it is more economical and more practical to correct adverse subsurface drainage conditions before construction rather than to attempt to handle this situation as a maintenance operation" (Smith, 1964). Some drainage facilities, such as pervious blankets or stabilization trenches, cannot be installed after the embankment has been constructed. Even when the unanticipated groundwater is discovered at the outset of construction, extra costs are likely to develop because of the need for extra materials and resulting construction delays. These economic considerations emphasize the importance of a thorough study of existing and potential groundwater conditions during the preliminary field investigations for the soil survey report.

In relatively flat terrain with high groundwater tables, many problems can be eliminated by ensuring that the grade line is several feet above natural ground. Such grade lines frequently require the abandonment of the concept of balanced earthwork. However, the advantages in relation to drainage and maintenance outweigh the advantages of balanced design. In addition, for low embankments in some northern regions, pervious blankets are used to cut off capillary rise into the embankment and thus to reduce the potential for frost heave.

As the terrain becomes more hilly or mountainous, groundwater problems are likely to become more severe. In addition, the interruption of natural surface runoff must



DETAIL FOR TRANSVERSE UNDERDRAIN CUT TO FILL CONDITION



DETAIL 1

Figure 9. Typical section at cut-fill transitions.

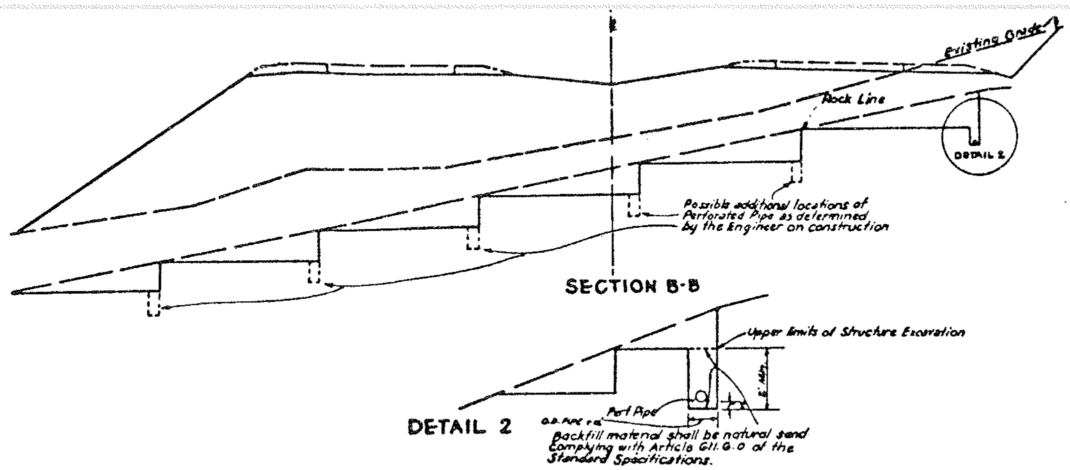


Figure 10. Typical detail for longitudinal underdrain.

be considered. Furthermore, in these locations major subsurface drainage problems can occur in relatively impervious fine-grained soils with fissures, joints, or bedding planes. These conditions do not lend themselves to the use of rigorous analytical solutions for calculating quantities and paths of seepage.

The most commonly used subdrain systems consist of pervious blankets and/or perforated pipe subdrains. These subsurface drainage facilities may be more commonly required in cuts than in fill areas. With respect to embankments, the most common need for subdrain systems will be at cut-fill transitions and in sidehill locations. The specific design of underdrain systems and pervious blankets may vary considerably. Both longitudinal and transverse subdrain systems are used, depending on the topography. Typical subdrain provisions used by one agency at cut-fill transitions are included in Figure 9. Similar installations in sidehill fills are shown in Figure 10. Sometimes longitudinal drains are installed in trenches in cut or natural slopes above fills to intercept groundwater moving down the slope. Whenever pipe drains are installed in relatively impervious materials, it is essential to provide a bed of permeable material for the pipe and to backfill the excavation with the same material.

When large quantities of water are involved, a continuous pervious blanket may be more economical than a pipe drain system. Pervious blankets usually are at least 1 ft thick, and careful consideration must be given to the gradation of the pervious material. The gradation specifications must ensure that the material is coarse enough to be free draining, yet not so coarse as to permit migration of the fine-grained soil into or through the permeable material. Much has been written on the design of filter systems; and although much of this literature is directed toward filters for earth dams, the concepts generally are applicable to highway drainage problems. These design procedures generally relate the grain size characteristics of the filter material to those of the adjacent soil. One commonly used requirement, shown in Figure 11, is that the 15 percent particle size of the filter should be at least five times larger than the 15 percent size and no more than five times larger than the 85 percent size of the adjacent soil layer.

In hilly or mountainous terrain, the method of subsurface drainage will depend on the topography and the depth at which water-bearing strata are encountered. This may be illustrated by considering the variety of drainage methods employed in California. When the water-bearing material is relatively shallow (at depths of less than 10 or 20 ft) and underlain by firm material, the soft wet material may be excavated and replaced with a pervious blanket. When the subsurface water is encountered at depths of 10 to 30 or 40 ft, stabilization trenches, described in the preceding section, may be employed. Occasionally, horizontal drains are used to remove subsurface water at greater depths than is economically practical by stripping or trenches. These drains are installed by drilling laterally from a point at or below the toe of the proposed fill slope. Such drains, which may be installed to lengths of 150 to 300 ft, can reach well beyond the toe of the uphill slope. Sometimes additional drains are installed from the toe of the uphill slope to intercept subsurface water before it reaches the embankment. Usually collection systems are required to remove the outflow from the horizontal drains.

Groundwater at great depth can be intercepted by installing vertical relief wells in conjunction with horizontal drain systems. The primary purpose of the vertical wells is to relieve hydrostatic pressures rather than to provide for complete drainage. Relief wells have been installed to depths of 40 ft. Six-inch-diameter perforated pipe is placed in the center of a 24-in. hole, which is then backfilled with

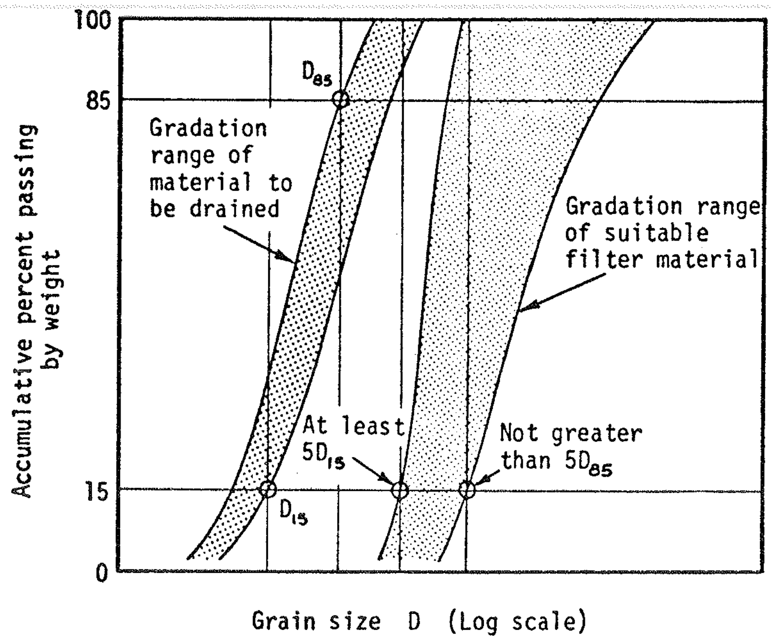


Figure 11. Typical gradation requirements for filters and pervious blankets.

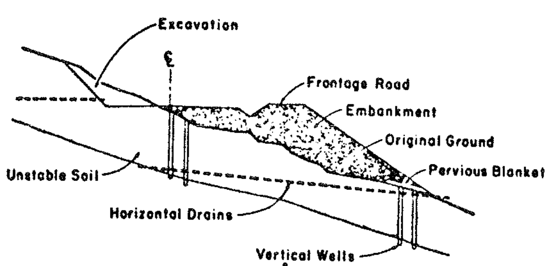


Figure 12. Cross section of typical vertical wells and horizontal drains.

permeable material. Horizontal drains are installed to intercept the relief wells. A cross section of a typical relief well and horizontal drain system is shown in Figure 12.

Subsurface drainage methods, which vary widely in cost and complexity, are available for the control of almost any groundwater condition. The major problem is the identification of the severity of the groundwater conditions at specific sites so that the most economical and effective subdrainage system can be employed. The importance of an adequate site investigation of the groundwater conditions cannot be overemphasized.

CHAPTER FOUR

DESIGN OF FILLS

DESIGN CONSIDERATIONS

In practice, the design of highway fills generally consists of establishing the height and the side slopes of the embankment and specifying criteria for placement of the fill. The placement criteria generally are included in each agency's general specifications and are discussed in Chapter Five. The factors used to establish the size and shape of the embankment are discussed briefly in the following paragraphs.

The height of an embankment depends on the proposed highway grade line, which is established primarily on the basis of geometric design criteria and secondarily on the basis of balanced earthwork. Only rarely does the strength or compressibility of the fill or the underlying foundation material influence the design height. Standard side slopes are established to satisfy safety standards and to facilitate maintenance. Typical standard slopes established on the basis of such criteria are shown in Figure 13. Current trends indicate that the use of 2:1 as a standard design slope is common. Design engineers ordinarily will assume that the standard design slopes will be stable unless stability problems and alternate slope recommendations have been indicated by the soils engineers in the soil survey for the project. When such recommendations are included in the soil survey, they usually are incorporated into the design. Thus, except in cases of poor foundation conditions, described in Chapter Three, the engineering properties of the fill and underlying foundation soils have relatively little influence on the design geometry of the fill.

Balanced earthwork is practiced when it can be accomplished without violating geometric design criteria. However, strict adherence to balanced earthwork design can lead to serious construction and maintenance problems by encouraging the use of poor quality soils from cut sections and the use of locations with poor foundation conditions. To accomplish a balanced design, it is necessary to estimate the shrink and swell factors for the excavated materials and to estimate the quantities of poor quality materials that should be wasted. In areas of relatively flat topography, balanced design appears particularly undesirable because the grade line would be established very close to natural ground level, with frequent transitions between small cuts and low fills. Under such circumstances, construction of a uniform subgrade is made more difficult. Furthermore, surface and subsurface drainage problems may be introduced, and winter maintenance may be made more difficult in northern regions. For these reasons, many engineers now prefer to ignore balanced earthwork concepts and to construct continuous low embankments over relatively flat terrain.

The design load used to evaluate the stability and deformation of an embankment is the weight of the overlying embankment and pavement materials. Except for the upper few feet, embankments are not seriously affected by traffic loads. In current practice, the density rather than the strength or compressibility is specified for fill materials. When fill is compacted in accordance with current density requirements, its compressibility will be negligible. Hence, except for very high fills, the compressibility of compacted materials is not generally evaluated. Similarly, the strength of compacted materials generally is assumed to be adequate to maintain the standard design slopes. As discussed in Chapter Two, stability and settlement considerations primarily concern the behavior of the foundation material beneath the fill. Current field experience tends to justify the assumption that for current standard design slopes and compaction requirements, the strength and compressibility of the compacted fill usually is not a problem.

Exceptions are noted in the case of high embankments, for which the strength and compressibility of the fill materials must be given special consideration. Most agencies define high embankments as those exceeding 40 or 50 ft. In the construction of high embankments, sometimes low-quality materials must be wasted, and in other instances zoning of material must be practiced. When zoning is necessary, the higher-quality materials are placed in the bottom of the embankment where the loads are the largest. Lower-quality materials are used in the upper parts of the embankment and capped with a layer of select material, which will serve as the subgrade subjected to traffic loads. Problem soils sometimes are placed in the center or lower portions of an embankment. For example, expansive soils will perform more satisfactorily deep within an embankment where they are better protected from drastic moisture

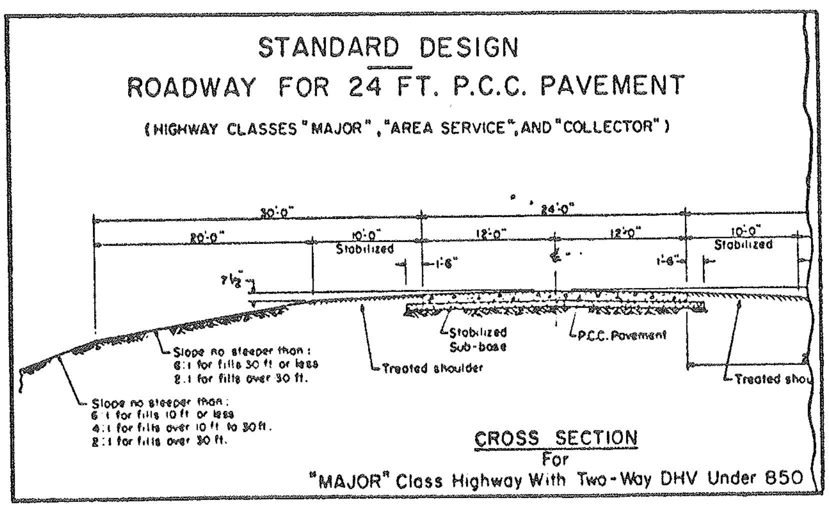


Figure 13. Typical standard design slopes.

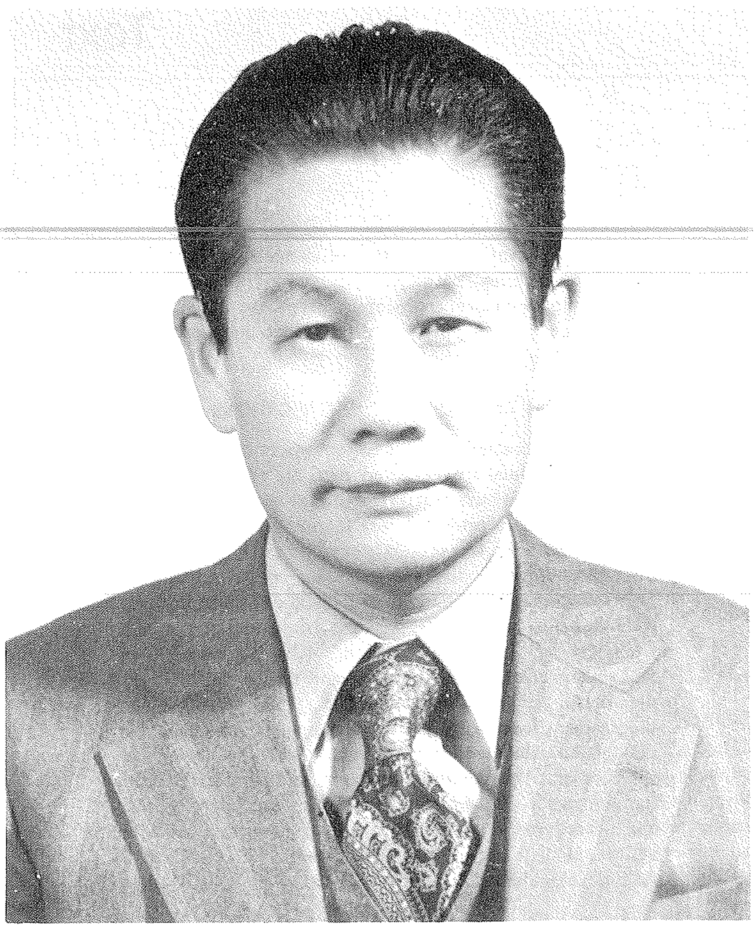
changes and their tendency to swell is inhibited by the weight of the overburden. Similarly, micaceous soils may be placed where they will not be subjected to traffic loads.

The strength and the compressibility of the fill must be considered when low-quality materials must be used or soils must be placed under adverse conditions. This situation most commonly arises when extremely wet fine-grained soils must be used. Recognition of such potential situations in the soil survey and consideration of them in the embankment design will minimize problems and delays during construction. For example, if plan notes indicate that certain soils will be troublesome when wet, the construction division and the contractor may be able to schedule construction so as not to expose these soils during rainy seasons. In other instances, plan notes or special provisions may indicate special zones in which wet soils may be placed or special construction procedures (e.g., sandwiching with good materials).

The design of earth embankments is more significantly influenced by geometric design criteria, safety standards, and maintenance requirements than by the strength and the compressibility of the embankment and its foundation. When stability and settlement are the dominant factors, the embankment design more commonly is influenced by the properties of the foundation than by those of the fill material. In other words, current procedures and specifications for placement of fill are producing compacted materials with adequate engineering properties for current standard designs, except for the special situations outlined in the preceding paragraphs.

PREPARATION OF PLANS AND SPECIFICATIONS

Construction practices for the improvement of foundations for embankments are reviewed in Chapter Three. It is emphasized that the need for these construction procedures must be identified during the soil investigations and incorporated into the project design. In the preparation of design drawings and specifications, special treatments for embankment foundations should be indicated in sufficient detail by means of special provisions and plan notes. Complete subsurface information, including boring logs and soils profiles, should be presented on the construction drawings. Similarly, the need for special embankment placement procedures, such as zoning of materials or anticipated placement under adverse conditions, should be specified clearly on the plans. Failure to adequately inform the contractor of subsurface conditions, special problems, and procedures when contracts are let inevitably leads to additional costs and delays when the unanticipated conditions develop during construction.



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Project Correspondent In-Joon Chung, College of Engineering, Seoul National University, Korea.

Conference Arranged by The Department of Civil Engineering and
The Tennessee Highway Research Program
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University Center, The University of Tennessee

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45th Annual Tennessee Highway Conference

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Knoxville

CONTENTS

LOCATING GROUND WATER FOR DESIGN OF SUBSURFACE DRAINAGE IN ROADWAYS AND EMBANKMENTS

J. A. TODD

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A recent entry in one of our project diaries reads as follows: "4½ inches of rain occurred last night and yesterday; it is still raining today on project. A fill failed at Station 80+00. All of the embankment slipped out leaving bare rock in the bottom of the scour. A small stream of water is flowing from a crevice in the rock. No water has been noted in this area previously."

Such slides are not unusual in steep mountainous terrain. The fill failure was caused by excessive rainfall, and intrusion of water into the original ground beneath the embankment. Now what about that small stream in the hollow? There is no water running there now. Suppose we go ahead and cut a few slope benches and reconstruct the fill and forget all about that little stream of water; what will happen when it rains again? If the point where the project engineer saw water issuing is plugged up and it rains hard enough long enough, that fill will slide off the side of the mountain again.

Now let us go into my subject, locating ground water for designing subsurface drainage in roadways and embankments. I will begin by talking about springs.

We are concerned with three types of springs: 221

(1) Fissure springs, so called because underground water passages are located in cracks, crevices, and fissures in solid rock.

(2) Tubular springs, so called because underground water passages are tubular in shape. These springs are found in soil drift or water soluble rock.

(3) Seepage springs, so called because the issue occurs in a zone of porous material and there is no bold or turbulent spring point. The underground water passages for seepage springs usually occur in a layer of pervious material superimposed on less pervious material such as clay, rock, or hardpan.

All three types of springs described above are quite common in this area. It is generally known that springs often dry up in periods of sustained dry weather and that many will flow only after periods of excessive rainfall. Springs that dry up in dry weather and flow in wet weather are commonly called "wet-weather springs." Springs that continue to flow regardless of rainfall are called permanent springs.

It is easy to locate permanent springs, but how does one locate a wet-weather spring? Frankly, it is not easy, unless the engineer makes frequent trips into the area in question during periods of excessive rainfall. It is not always possible for the engineer to do this. Therefore, he must rely upon surface features which generally are associated with springs. The best way to learn to identify these surface features is to visit active springs in the general area of the site to be investigated and note the following:

(1) Where does this spring occur?

Usually a spring will occur at an abrupt change in the slope such as the base of cliff, bottom of a hill, or in a hollow or draw.

(2) What occurs below the spring?

Usually a spring run is evident below the spring point. In some cases, there may be a deep gully-like channel. In other cases, there may be a delta-like silt fan below the spring point.

(3) What other conditions may be observed at the spring?

One would expect to find certain water-loving plants such as willows, water cress, bullrush or moss in the spring area or along the spring run. Also, one may observe the lack of leaf mold accumulation and crayfish castings in the spring area.

After making these observations at permanent springs, it will be easy to locate areas where wet-weather springs might occur. There is generally but one difference. In the wet weather spring area there may be no flow of water. Generally some or all of the other features previously stated will be present.

During the course of constructing a highway, excavations will be made and embankments will be constructed from excavated material to obtain the grade profile. These excavations will no doubt encounter many underground water passages. There will be a flow of water if these underground passages feed permanent springs, and there may or may not be a flow of water if wet weather springs are fed by these water passages. A good percentage of cut slope failures occur where underground water passages are encountered. This condition, I feel sure most of you have observed. It is reasonable to assume that underground water passages may exist in areas where embankments are to be constructed. When there are no active springs in the embankment area locating underground water passages presents a problem. They can be located if the engineer will make a close inspection of the back slopes of pioneer roads, haul roads and slope benches constructed in the embankment area.

The engineer must know what he is looking for, otherwise, he may not recognize inactive water passages. Now let us go back to our spring type. There are three of them (1) fissure springs, (2) tubular springs, and (3) seepage springs. Each type of spring is identified by the underground water passages.

In fissure springs, the water passages are cracks, crevices, or fissures in solid rock strata. Suppose a pioneer road into an embankment area is blasted into solid rock and the back slope reveals numerous cracks, crevices or fissures in the solid rock, how can one tell whether they serve as underground water passages. There are several ways to determine whether water has actually flowed in the cracks and crevices. (1) There will be a noticeable difference in color in the cracks, crevices or fissures which serve as water passages. (2) In vegetated areas tiny feed roots of plants will be found in the cracks and crevices. Where water is frequently present in cracks or crevices the tiny feed roots are usually white in color and the root hairs show no evidence of soil film. (3) The larger cracks, crevices or fissures will usually contain sand, or fine gravel in the areas where passages are enlarged. (4) The normal cracks and crevices may be filled with clay or silt in all areas except where water passage occurs.

Now let us consider tubular water passages and field methods of identification. As stated previously, tubular water passages are located in soil drift or water soluble rock. Usually these passages are close to the surface of the ground when they are in soil drift, above the zone of solid rock except in the case where the solid rock is soluble and contains water passages. In the root zone of cover vegetation the tubular water passages are likely to be identified as a root hole. If the passage is carefully excavated for a short distance, one will observe that the passage will continue to be singular and minor deposits of sand or gravel will be noted. The size of tubular passages will vary considerably. In sandy soil or weathered rock, passages up to 2 feet in diameter have been observed. Generally, the passages that feed wet-weather springs are less than 3 inches in size. In water soluble rock the passages will often enlarge into huge caverns. Many caves are formed in this manner. It is not likely that the experienced engineer will fail to recognize the tubular water passages in water soluble rock or weathered rock unless they are destroyed by blasting or other construction activities.

Seepage water passages are, as previously stated, located in layers of pervious material superimposed on less pervious material. The layers of pervious material may be either talus slopes with rock fragments up to several cubic yards in size, or fine sand. The thickness of the layers will vary considerably. Where active seepage springs are found during periods of dry weather, water passages are usually found in thin pervious layers that slope very gently. Thick layers of pervious material above layers of less pervious material are prime suspects for water passages that feed wet-weather springs. As in the case of fissure springs there will be noticeable color changes where a flow of water has occurred. Also, the white feed roots of cover vegetation will be noted.

We are now able to recognize underground water passages disturbed by construction operations. When the material excavated for embankment construction is porous and the methods of placing is such that a pervious embankment will result, there is no reason for the engineer to install mechanical means for providing drainage for the disturbed water passages if none are contained in thick layers of pervious material, water soluble rock or weathered rock. If the excavated material to be used for embankments is impervious and/or the methods of placement result in a very dense embankment, then drainage must be provided for all disturbed water passages, otherwise, embankment failures will occur. If hard rock fragments are used for embankment construction, drainage for small springs can be effected through the voids in the rock. I might point out that certain types of rock weathers on exposure to air and water and eventually the voids will silt up and block water passages. Rock fragments containing iron and copper pyrites are likely to react chemically on exposure to air and water, and serious settlement will result.

Design of subsurface drainage for areas where embankments are to be constructed is very simple once the water passages are located. The only factor that must be remembered is that water will not flow uphill. The use of small diameter perforated pipes or porous material in trenches will be satisfactory in most cases. The engineer can estimate the amount of water that will occur on the basis of area of water passages. The seepage passages will be hardest to estimate. It is much more economical to overestimate than it is to reconstruct embankments.

After the highway is completed or partially completed, failures may occur in embankments and subgrades due to excessive soil moisture. The location of sources of water which contribute to the failure which occurs after construction has started, is rather difficult as compared to methods previously described. The issue point is not usually visible or accessible without exploratory excavations. In most cases, the corrective measures are simple once the source of water has been located.

Since 1957 a new method of identifying sources of water has been employed in the Gatlinburg, Division of Region 15, Bureau of Public Roads. This method, which I will describe briefly, is still in the experimental development stage and data is not complete for publication.

This new method for tracing water sources involves the use of fluorescent dyes and portable ultra-violet lights or lamps. Fluorescent dye was first used on the Blue Ridge Parkway in North Carolina. In the beginning, the methods employed were more or less cut-and-dry. The methods used today have evolved on a one-thing-follows-another basis. We are still experimenting at every opportunity and improving our methods as we go.

To locate ground water where fills are failing the survey is conducted as follows:

(1) Make a map of the fill area, adjacent cuts and drainage area above the fill. On this map, show the original ground contours, construction limits, grade points, haul road diagrams, channel changes, existing drainage structures and slope benches constructed.

(2) Indicate on the map, soil conditions encountered in the adjacent cuts and soil types used to construct the fills. Also, in-place density data if tests were made on embankments.

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(3) In the area above the fill, locate all possible water sources using methods previously described for locating wet-weather springs.

(4) Determine from the map and other data, the areas which are suspected sources of water contributing to the failure. The following factors should be considered:

a. Active streams above the fill area. In the course of constructing cross drainage structures, the streams may be diverted from existing channels temporarily and seepage springs in the old channel may be covered up. If no drains are provided in the old channel, it is highly possible that water may enter the fill area from seepage springs in the old channel. The seepage springs in the old channel may be fed from the active stream above the fill area.

b. Haul roads cut for the contractor's operations. It has been noted in the field that at locations where haul roads were cut below the profile grade of the roadway and later backfilled with pervious material that ground water in the cut areas was diverted into the fill area.

c. Standing water in ditchlines on the uphill side of cuts adjacent to the fill area. Due to haul roads or porous embankment material, surface water impounded in ditches may seep downward into cracks and crevices in blasted rock and via this media, enter the fill area and cause failures.

d. Existing drainage structures. Underdrain pipe constructed in cut areas may actually collect water and divert this water via cracks and crevices in blasted rock into fill area. Also, cross drain pipe culverts which cross haul roads may actually leak and contribute water to a fill area.

e. Porous layers of material placed in embankments. In many instances where part-width fill construction is performed, porous pockets may result in embankments and actually perch water in the embankment. Surface water may seep into the upper layers of the fill and saturate the lower layers.

Now that we have determined the areas of suspicion, the next problem is to identify the actual source of water. This is done by use of fluorescent dyes in the following manner:

Put a quantity of dye in the active streams over a period of time, say 10 to 30 minutes. This dye, easily seen in daylight, will quickly color the water downstream from the point where it is introduced. If the flow of water in the fill failure is enough to fill a 2-inch water pipe, the dye will show up in the fill area in 30 to 40 minutes from the time it is introduced at the source. If the flow of water in the fill area is slight, it will take more time and the color will partially filter out making it impossible to detect under natural light. In such cases, the water is sampled at intervals and tested for fluorescence in a dark chamber, or the survey may be conducted at night. When the soil mass between the point of introduction of the dye at the source and the issue point in the fill is very dense, we have noted coloration as much as 3 to 4 days after the dye was placed. If no dye shows up in the fill area after a few days, then the source first suspected will be eliminated. A cache of dye is then placed in another area of suspicion and the follow-up sampling of water and testing for fluorescence will either confirm the source or eliminate it, and so on until all areas of suspicion have been checked.

The engineer must try to avoid use of the same color dye in more than one area of suspicion. Suppose uranine dye is used in 4 different places and finally dye shows up in the fill failure area. Where did the water come from? Any one of the 4 places. We have experimented with dyes and find we have only 2 dyes that can be distinguished in weak solution and 4 dyes that can be distinguished in medium to strong solution by use of ultra-violet light. The dyes we have used in our work are non-toxic.

We have used fluorescent dyes to locate sources of water where subgrade failures occur on old roadways. The survey is conducted in the same manner as previously described. Usually due to the lack of stream flow except in periods of excessive rainfall we cache the dye in areas of suspicion several months and check the wet area after each rain in darkness until fluorescence occurs. Then we design corrective measures on future paving projects.

We have used fluorescent dyes to trace water discharged from drainage ditches and structures when property owners complain of damage. In one area, several tort claims were settled after we proved the sources of water with fluorescent dyes.

At the present time, we have several surveys in progress where embankments have settled or cut slopes have failed. I might add that we have not been 100% successful in all of our surveys with fluorescent dye.

I have with me several small samples of fluorescent dye; also, some dye crackers which may be of interest. I also have three portable ultra-violet lamps which we have used in our surveys.

The dyes I have with me are numbered from 1 through 6. Dyes numbered 1 through 5 were purchased from Leeben Color and Chemical Company, 103 Lafayette Street, New York 15, N. Y. Dye No. 6 was purchased from Industrial Colloids, Knoxville, Tennessee.

Dye No. 1 is red in natural daylight. It fluoresces brown under long-wave ultra-violet light; it fluoresces olive drab under short-wave ultra-violet light. This dye is identified by the producer as No. LS671.

Dye No. 2 is blue in natural daylight. It fluoresces green under long-wave or short wave ultra-violet light. This dye is identified by the producer as No. LS665.

Dye No. 3 is violet in natural daylight. It fluoresces green under long-wave or short-wave ultra-violet light. This dye is identified by the producer as No. LS673.

Dye No. 4 is red in natural daylight. It fluoresces bright orange under long-wave or short-wave ultra-violet light. This dye is identified by the producer as DC Red No. 19.

Dye No. 5 is black in natural daylight (this dye must be used in heavy concentration, otherwise it is not recognizable). It fluoresces dark green under long-wave or short-wave ultra-violet light. This dye is identified by the producer as No. LS663.

Dye No. 6 is yellow in natural daylight. It fluoresces yellow green under long-wave ultra-violet light and yellow under short-wave ultra-violet light. This dye is called Uranine Supra No. 59693.

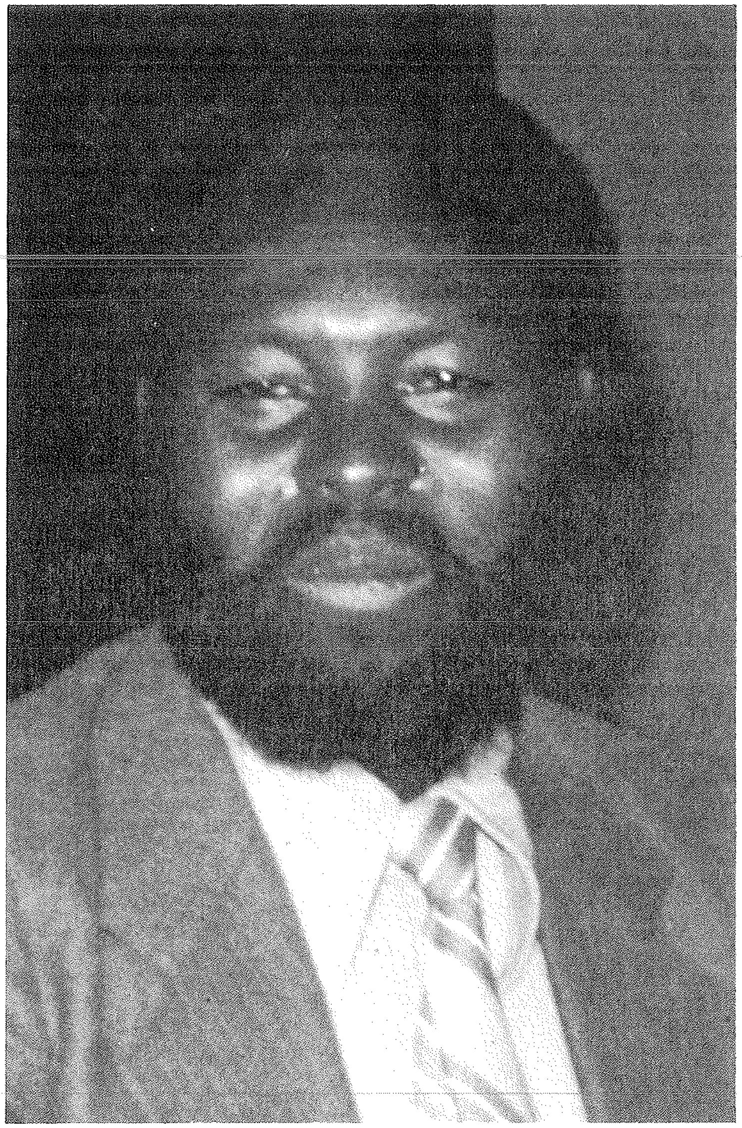
We have mixed dye No. 4 and dye No. 6 in equal parts to produce another easily identified dye. This dye is green in natural daylight and fluoresces bright yellow under long-wave or short-wave ultra-violet light.

The dye crackers on exhibit were made by adding wheat flour in equal part by volume to dye crystals and wetting to form a stiff paste. This paste is spread on metal sheets and dried in the sun to produce the crackers. The dye crackers do not dissolve as quickly as the dye crystals and contamination from a cache may be had over a longer period of time. The dye crackers were developed for this reason.

We have also discovered that soil wet with dyed water and oven dried will fluoresce when pure water is added later. We are now planning to investigate the capillarity fringe above the ground water table to see if we can contribute any data in this field of research.

The three portable ultra-violet lights on exhibit were purchased from Ultra-Violet Products, Inc., 5114 Walnut Grove Avenue, San Gabriel, California. The M-12 portable ultra-violet light is battery powered and very efficient. Short-wave ultra-violet rays are emitted by this unit. Short-wave ultra-violet rays are dangerous to the naked eye and should be handled very carefully. The UVS-12 is portable when operated from a 6-volt hot-shot battery using a converter. It also operates from a 110-volt AC outlet with ordinary plug-in cord. This unit also emits short-wave ultra-violet rays which are dangerous to the naked eye. The UVL-22 is a long-wave ultra-violet light. This unit is not dangerous to the naked eye. The fluorescence produced by this light is somewhat brighter than that produced by the short-wave ultra-violet rays. It is portable when operated from a 6-volt hot-shot battery using a converter.

I have discussed location of ground water in three stages: Before any construction activity has taken place, after pioneer roads and slope benches in fill areas have been constructed, and finally during construction or after completion of roadways. The fact that most of my paper emphasizes methods of correcting ground water found during construction does not obviate the need for thorough field investigations in the preliminary design to avoid problems caused by ground water.



Project Correspondent Francis J. Gichaga, Chairman, Civil Engineering Department, University of Nairobi, Kenya.

Bibliography

The following bibliography contains two sets of references. The first set consists of a reference for each selected text that appeared in the preceding part of this compendium. The second set consists of references to additional publications that either were cited in the selected texts or are closely associated with material that was presented in the overview and selected texts. Each reference has five parts that are explained and illustrated below.

(a) Reference number: This number gives the position of the reference within this particular bibliography. It is used in the compendium index but should ***not*** be used when ordering publications.

(b) Title: This is either the title of the complete publication or the title of an article or section within a journal, report, or book.

(c) Bibliographic data: This paragraph gives names of personal or organizational authors (if any), the publisher's name and location, the date of publication, and the number of pages represented by the title as given above. In some references, the paragraph ends with an order number for the publication in parentheses.

Bibliografía

La siguiente bibliografía contiene dos series de referencias. La primera serie consiste en una referencia para cada texto seleccionado que apareció en la parte anterior de este compendio. La segunda serie consiste en referencias a publicaciones adicionales que fueron mencionadas en los textos seleccionados o que se asocian íntimamente con el material que se presentó en la vista general y los textos seleccionados. Cada referencia tiene cinco partes que se explican y se ilustran abajo.

(a) Número de referencia: este número indica la posición de la referencia dentro de esta bibliografía en particular. Se utiliza en el índice del compendio pero ***no*** deberá utilizarse al pedir publicaciones.

(b) Título: el título de la publicación completa o el título de un artículo o sección dentro de una revista, informe, o libro.

(c) Datos bibliográficos: este párrafo da los nombres de autores personales u organizacionales (si hay alguno), el nombre del editor y su dirección, la fecha de publicación, y el número de páginas representadas por el título en la parte (b). En algunas referencias el párrafo termina con un número de pedido para la publicación en paréntesis.

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(b) Titre : cela indique ou le titre du livre entier, ou le titre d'un article ou d'une section d'une revue, un rapport, ou un livre.

(c) Données bibliographiques: ce paragraphe indique les noms des auteurs personnels (quand il y en a) ou des auteurs collectifs (organisation), le nom de l'éditeur et son adresse, la date de l'édition, et le nombre de pages qui sont incluses sous le titre dans (b). Certaines références se terminent par un numéro entre parenthèses qui indique le numéro de commande.

(d) Availability information: This paragraph tells how the referenced publication is available to the reader. If the publication is out-of-print but may be consulted at a particular library, the name of the library is given. If the publication can be ordered, the name and address of the organization from which it is available are given. ***The order should include all information given in parts (b) and (c) above.***

(e) Abstract: This paragraph contains an abstract of the publication whose title was given in part (b).

(d) Disponibilidad de la información: este párrafo indica la disponibilidad al lector de la publicación referenciada de una de dos formas como sigue. (1) La publicación está agotada pero puede ser consultada en la biblioteca indicada, donde se sabe que se posee una copia, o (2) la publicación puede ser pedida de la organización cuyo nombre y dirección están indicados. ***El pedido deberá incluir toda la información dada en las partes (b) y (c).***

(e) Resumen: este párrafo es un resumen de la publicación cuyo título se dió en la parte (b).

(d) Disponibilité des documents: ce paragraphe indique les deux façons dont le lecteur peut acquérir les documents: (1) L'édition est épuisée, mais une certaine bibliothéque détient ce document et il peut être consulté. (2) Le document peut être commandé à l'organisation dont le nom et l'adresse sont indiqués ici. ***L'ordre de commande doit inclure toutes les informations données dans les parties (b) et (c).***

(e) Analyse : ce paragraphe est une analyse du texte dont le titre est cité dans la partie (b).

Illustration (from Comp. 1) **Ilustración** (del Comp. 1) **Illustration** (du Recueil 1)

(a) Reference number
(a) Número de referencia
(a) Numéro de la référence

(b) Title
(b) Título
(b) Titre

(c) Bibliographic data
(c) Datos bibliográficos
(c) Données bibliographiques

(d) Availability information
(d) Disponibilidad de la información
(d) Disponibilité des documents

(e) Abstract
(e) Resumen
(e) Analyse

The order should include all information given in parts (b) and (c) above.

El pedido deberá incluir toda la información dada en las partes (b) y (c).

L'ordre de commande doit inclure toutes les informations données dans les parties (b) et (c).

Reference 5
A REVIEW OF HIGHWAY DESIGN PRACTICES IN DEVELOPING COUNTRIES

Cron, Frederick W. Washington, DC: International Bank for Reconstruction and Development; 1975 May. 57 p.

Order from: International Bank for Reconstruction and Development, 1818 H Street, N.W., Washington, DC 20433.

The design standards of some 150 highway projects financed by the International Bank for Reconstruction and Development between 1960 and 1970 are reviewed, and areas of agreement between the standards of the 63 countries studied are identified; practical highway standards based on these areas of agreement are sketched for the guidance of planners in developing countries. The roads discussed here, fall into three functional categories: a small group of expressways, freeways and toll roads carrying large volumes of traffic; a very large group of 2-lane highways carrying a wide range of traffic volumes serving both local and long distance traffic; and a smaller group of low-traffic tertiary or special purpose roads existing primarily for land service. Comments are made on the problem of classifying highway standards, and on the comparison of standards. Conclusions regarding standards for the capacity- related elements of design and standards for the velocity-related elements of design (radius of curvature, stopping sight distance, passing sight distance) are discussed, as well as the horizontal and vertical clearances for bridges. The standard live loadings for bridges, the structural capacity of pavements and legal load limits are covered, and conclusions relating to pavement design, design standards for 2-lane highways, incremental development of highways, and levels of service are presented.

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Schuster, Robert L; Krizek, Raymond J., eds. Washington, DC: Transportation Research Board; 1978; 234 p. (Special Report 176).

Order from: Transportation Research Board, Publications Office, 2101 Constitution Avenue, N.W., Washington, DC 20418.

This volume brings together, from a wide range of experience, such information as may be useful in recognizing, avoiding, controlling, designing for, and correcting landslide movement. Current geologic concepts and engineering principles and techniques are introduced, and both the analysis and the control of soil and rockslopes are addressed. New methods of stability analysis and the use of computer techniques in implementing these methods are included. Rock slope engineering and the selecting of shear-strength parameters for slope-stability analyses are covered in separate chapters. The first part of the book deals primarily with the definition and assessment of the landslide problem. It includes chapters on slope-movement types and processes, recognition and identification of landslides, field investigations, instrumentation, and evaluation of strength properties. The second part of the book deals with solutions to the landslide problem. Methods of slope-stability analysis, design techniques and remedial measures that can be applied to both soil and rock slope problems are included.

Reference 2
SOIL MECHANICS IN ENGINEERING PRACTICE

Terzaghi, Karl; Peck, Ralph B. New York, New York: John Wiley & Sons, Inc. 1967; 729 p. (2nd Ed).

Order from: John Wiley & Sons, Inc. Publishers, 605 Third Avenue, New York, New York 10016.

This book consists of three parts (A, B, and C). Parts A and B cover the basic aspects of the physical properties of soils and theoretical soil mechanics. Part C explores techniques for obtaining satisfactory results in earthwork and foundation engineering at a reasonable cost in spite of the complexity of the structure of natural soil strata and the inevitable gaps in practical knowledge of soil conditions. Almost every practical problem in this field contains some features without precedent and every engineer must use all available methods and resources with careful discrimination. In Part C, practical problems are discussed, starting with a critical survey of conventional methods and proceeding step by step to whatever improvements have been realized in the area of soil mechanics research. A semiempirical approach is adopted in this section, and a list of references is included in each chapter. The various chapters of Part C are as follows: soil exploration (purpose, methods and program); earth pressure and stability of slopes (retaining walls, drainage, lateral supports in open cuts, stability in open cuts, soil compaction, design of fills and dikes, stability of base of embankments); foundations (foundations for structures, raft, pile and pier foundations); settlement due to exceptional causes (settlement due to construction operations, lowering the water table, and vibrations); dams and dam foundations (earth, rockfill, concrete, and supervision during construction); and performance observations (scope, measurement of displacements, earth pressures and porewater pressures, and records of observations). A French translation of this book was published by Dunod, 24–26 Boulevard de l'hopital, 75006 Paris, France, in 1961. A Spanish translation of this book was published by "El Ateneo" Pedro García, S.A., Librería, Editorial e Inmobiliaria, Florida 340, Buenos Aires, Argentina, in 1958.

La quatrième édition de ce livre a été traduite en français et publiée en 1961 par Dunod, 24–26 Boulevard de l'hopital, 75006 Paris, France.

Este libro ha sido traducido al castellano e impreso en 1958 por "El Ateneo" Pedro García, S.A., Libreria, Editorial e Inmobiliaria, Florida 340, Buenos Aires, Argentina.

Reference 3
HIGHWAY SOILS ENGINEERING: 11 REPORTS

Highway Research Board, Washington, DC; 1971; 126 p. (Highway Research Record Number 345).

Order from: University Microfilms International, 300 North Zeeb Road, Ann Arbor, Michigan 48106.

Information useful to persons concerned with the structural design and performance of a roadway and that would contribute toward understanding the concepts of pavement action, reaction and performance are presented in the first five reports of this publication. The first three papers present concepts of criteria using deflection and curvature, resilient response, and dynamic pulse times. The fourth paper (Analysis of Stresses and Displacements in Three-Layer Viscoelastic Systems) presents a highly theoretical approach to the analysis of stress and displacements, and the fifth paper makes a valuable contribution in noting the difference between stresses and displacements under assumed and actual tire pressures. In the sixth paper, a new method for determining the tensile strength of soils by using a double punch test is described. In this paper, fundamental relationships are developed, and comparisons with other tensile tests are discussed. The seventh paper describes the applicaiton of probability functions to uncertainty considerations and cost consideraitons to obtain a rational procedure for determining safety factors associated with the bearing capacity of cohesive soils. Practical "chart" solutions for slope stability problems are compiled in the eighth paper. These cover a wide variety of conditions and can be used in rapid investigation of preliminary designs. The ninth and tenth papers describe the special design of a 383 ft-high earth embankment and its subsequent behavior. The papers also compare the design strength values with "as-built" strength values. The last paper (11th) describes the determination of stress distribution around circular tunnels by using a conformal mapping technique that applies complex variables to this special stress analysis problem.

Reference 4
SLOPE DESIGN GUIDE

United States Department of Agriculture, Forest Service, Portland, Oregon; February 1973; 27 p. (Transportation Engineering Handbook; FSH 7709.11).

Order from: United States Department of Agriculture, Forest Service (Engineering Soils and Materials Group Engineering), Region 6, P.O. Box 3623, Portland, Oregon 97208.

This guide, which provides general values for cut and fill slope ratios, is based on soil properties that are identified by the Unified classification system. Data needed to use the guide are soil classifications, general field conditions with respect to density and moisture, and height of cut or fill. The guide was developed from typical soil strength values by using chart solutions for slope stability, studies that use the conventional method of slices, published empirical relations, and the authors' experiences. Data used to develop the guide were related to the following: (a) the effect of seepage in coarse-grained materials determined by using one-half the angle of internal friction as the effective angle of internal friction for the high ground-water condition; (b) the angles of internal friction used for the development of the maximum slope ratios for sands and gravels with non-plastic fines; (c) soil strength values used for the development of charts for sands and gravels with plastic fines; and (d) soil strength values used for the development of charts for fine-grained soils. The guide considers homogeneous soils, stratified deposits, residual soils, cemented and special soils, embankments, benching, coarse-grained soils, fine-grained soils, and unweathered rock.

Reference 5
ANALYSIS OF LANDSLIDES

Highway Research Board, Washington, DC; 1952; 39 p. (Highway Research Board Bulletin 49).

Order from: University Microfilms International, 300 North Zeeb Road, Ann Arbor, Michigan 48106.

This bulletin presents two reports: (a) Determining Corrective Action for Highway Landslide Problems, and (b) Use of Field, Laboratory and Theoretical Procedures for Analyzing Landslides. The first paper, which considers landslide problems in unconsolidated materials, discusses basic fundamentals of landslide analysis, suggests a classification of corrective measures, and discusses in some detail the preliminary analysis of a landslide, data obtained from field study, and stability analysis. Appendices are included that cover typical stability analysis, elimination methods, and control methods—retaining devices, and direct rebalance of ratio between resistance and force. The second paper describes a study of data from three actual landslides. The study was confined to a two-dimensional analysis of a shear-type failure in shallow deposits of unconsolidated materials. The data were used to check the validity of the circular-arc method of slope analysis. Although the range of applicability of the method could not be established, when the data were combined with similar data from previous studies, the results indicated the limited applicability of this approach.

Reference 6
HANDBOOK ON LANDSLIDE ANALYSIS AND CORRECTION

Mehra, S.R.; Natarajan, T.K. eds. New Delhi, India: Central Road Research Institute; 1966. 114 p.

Order from: Central Road Research Institute, P.O. Box CRRI, New Delhi, India 110020.

Usable information based on past experience is presented in this handbook. It tries to define the limits within which the traditional techniques are applicable and suggests later techniques under circumstances in which traditional procedures are not as effective. Guidelines are presented on the alignment of hill roads, as well as information useful in distinguishing between unsuitable ground and favorable ground for making cuts. The handbook also notes the prerequisites for the success of the different preventive and corrective measures. The various chapters of the book cover landslides and classification, analysis for slope stability in soil cuts, slope design in bed rock cuts, and characteristic features of landslides peculiar to different soil types. Field and laboratory investigations are also covered. Techniques for prevention and correction and basic rules for analysis for prevention and correction are detailed. A landslide analysis questionnaire for data collection is included. The geology of Northern Uttar Pradesh, Assam, Jammu and Kashmir and the Ladakh region of South India is also described.

Reference 7
CONSTRUCTION OF EMBANKMENTS

Highway Research Board, Washington, DC: 1971; 38 p. (NCHRP Synthesis of Highway Practice 8).

Order from: University Microfilms International, 300 North Zeeb Road, Ann Arbor, Michigan 48106.

Information is provided on the materials, design, and construction considerations that must be coordinated from site investigation through the completion of embankment construction. Practices for subsurface investigation of embankment areas, treatment of foundations, design criteria, specifications, construction, and quality control for embankments are covered. The fact that most embankment problems are the result of faulty placement of the embankment points to the need for adequate subsurface investigation of the foundation conditions at embankment sites. The route location phase should include provisions for considering sites where subsurface conditions could cause problems such as landslides, sidehill cuts and fills, abandoned mines, soft foundations, buried streams, and sanitary fills. Unusual conditions should be identified and corrective measures prescribed before start of construction. In the design of embankments, geometric design criteria, and safety standards usually have precedence over considerations such as soft foundations and poor materials. The design of high embankments should consider the quality of the fill materials. Although standard specifications require fill material to be placed in relatively thin lifts and compacted by rolling, there is a trend toward placing greater reliance on density requirements. Methods of determining in-place density are discussed. Problems with expansive clays and frozen soils are also discussed.

Reference 8
PROCEEDINGS OF THE 45TH ANNUAL TENNESSEE HIGHWAY CONFERENCE

The University of Tennessee, Department of Civil Engineering and the Tennessee Highway Research Program. Knoxville, Tennessee: Engineering Experiment Station, The University of Tennessee; April 18-19, 1963; 106 p. (Bulletin Number 29).

Out-of-print; may be consulted at U.S. Department of Transportation, Library Services Division, Room 2200, 400 Seventh Street, S.W., Washington, DC 20590.

The 17 papers presented at the conference covered a wide range of subjects. Highway Problems related to city, county, and state administrations were reviewed. Traffic planning problems, progress of the Interstate highway system, snow and ice removal problems, and subsurface drainage design were covered. Materials used in the construction of highways were covered in papers that focused on soils, soil-cement and calcium chloride used as an accelerating admixture in concrete and in preventive maintenance. The paper, "Locating Ground Water for Design of Subsurface Drainage in Roadways and Embankments" discusses the problem of failures of embankments built on sloping ground.

ADDITIONAL REFERENCES

Reference 9
EFFECTS OF WATER ON SLOPE STABILITY

Hopkins, Tommy C.; Allen, David L.; Deen, Robert C. Lexington, Kentucky: Kentucky Bureau of Highways; October 1975; 41 p. (Report # PB-263-860/AS).

Order from: National Technical Information Service, 5285 Port Royal Road, Springfield, Virginia 22161.

A brief state-of-the-art review of the effects of water on slope stability and of the techniques for analysis are presented. The effective stress principle and basic considerations of slope stability, including design factors of safety, are discussed briefly. The derivations and effects of seepage forces and rapid drawdown on effective stress are also presented. Various conditions of external loading produce changes in effective stress. These changes are discussed and limiting conditions that should be analyzed are mentioned. Limitations of total stress analyses are discussed in detail. It appears that, for soils having a liquidity index of 0.36 or greater (normally consolidated), the undrained shear strength gives factors of safety. For soils with a liquidity index of less than 0.36 (overconsolidated), the undrained shear strength gives factors of safety that are too high; but the strength parameters can be corrected by the empirical relation presented herein. Data also show that the difference between vane and calculated shear strength increased as the plasticity index and (or) the liquid limit increased. An empirical relation for correcting vane shear strength is presented. A discussion of effective stress analysis, including differences between peak and residual ϕ angles for normally consolidated and overconsolidated soils, is presented. The residual ϕ angle decreases logarithmically with increasing clay fraction. The "critical" state of a clay is also defined. Shear strength parameters of a clay tested in that state correspond to the theoretical strength of an overconsolidated clay that has undergone a process of softening. To test a clay in the critical state, it is suggested herein that the soil should be remolded to a moisture content equal to 0.36 times the plastic index plus the plastic limit. Water may cause unstable conditions in earth slopes due to changes in geometry. Erosion of the toe or the slope can induce damaging stress. Piping through heaving or erosion of subsurface layers can cause damage. Construction of side-hill embankments can cause damming, which results in a rise in the water table. Methods of water detection are also summarized. These include tracers, electrical resistivity, and water table observations (most successful). A discussion of ways to monitor water pressures, including the types and operations of piezometers, is given. Guidelines for the design of earth slopes are included.

Reference 10
A SURVEY AND EVALUATION OF REMEDIAL MEASURES FOR EARTH SLOPE STABILIZATION

Schweizer, Rudolph J.; Wright, Stephen G. Austin, Texas: The University of Texas at Austin, The Center for Highway Research; August 1974; 137 p. (Research Report 161-2F; Project 3-8-71-161, Cooperative Highway Research Program with Texas Highway Department, and U.S. Department of Transportation, Federal Highway Administration. Record #PB-244 626/8SL).

Order from: National Technical Information Service, 5285 Port Royal Road, Springfield, Virginia 22161.

The results of a literature survey undertaken to identify remedial measures that have been used to stabilize earth slopes are presented. In this review attention is directed to specific case histories and field conditions where the remedial measures were actually used. These measures include drainage of surface and subsurface water, restraint structures, elimination and avoidance of the slide area, benching and slope flattening and a number of special procedures, including electro-osmosis, thermal treatment, and addition of stabilizing admixtures. Of the procedures reviewed, drainage of surface and subsurface water appears to be the most widely and successfully used technique. However, the success of each measure depends to a large degree on the specific soil and groundwater conditions for the slope and the degree to which these are correctly recognized in an investigation and design.

Reference 11
AN ENGINEERING MANUAL FOR SLOPE STABILITY STUDIES

Duncan, J.M.; Buchiagnani, A.L. Berkeley, California: University of California, Department of Civil Engineering; March 1975; 83 p. (Geotechnical Engineering).

Order from: University of California, Department of Civil Engineering, Berkeley, California 94720.

This guide for slope stability studies describes the characteristics and critical aspects of various types of slope-stability problems, geologic studies and site investigation procedures, methods of designing slopes, (including field observations and experience), slope stability charts and detailed analyses, factors of safety, and methods of stabilizing slopes and slides.

The manual emphasizes simple routine procedures and gives references to resource material and to more advanced procedures where appropriate.

Reference 12
ROCK SLOPE ENGINEERING: REFERENCE MANUAL; 8 VOLUMES

Piteau, D.R. and Associates Limited, West Vancouver, British Colombia: January 1979; variable paying (Report # PB-80-103294).

Order from: National Technical Information Service, 5285 Port Royal Road, Springfield, Virginia 22161.

This reference manual has eight main parts. Part A (Engineering Geology Considerations and Basic Approach to Rock Slope Stability Analysis for Highways) considers properties and special factors applying to behavior under the influence of applied stress, analysis of rock slopes, geomorphological considerations, and basic factors and properties of structural discontinuities significant to slope stability. Part B (Methods of Obtaining Geologic Structural Strength and Related Engineering Geology Data) covers mapping, drilling, core orientation, borehole viewing, geophysical methods, airphoto analysis and terrestrial photogrammetry, and determination of properties of intact rock and discontinuities. Part C (Approach and Techniques in Geologic Structural Analysis) describes structural domains and their delineation, orientation data using stereographic projections, and the use of the cumulative sums technique in geotechnical analysis. Part D (Slope Stability Analysis Methods) describes plane, step-joint, toppling, wedge, and rotational shear failures, block flow, and probability or reliability analysis for rock slopes. Part E describes rock slope stabilization, protection and warning-instrumentation measures and related construction considerations. Part F covers controlled blasting types and procedures, preshearing, and blasting damage and vibration effects. Part G describes the detail line engineering geology mapping method. Part H reproduces chapter 9—Rock-Slope Engineering—of the Transportation Research Board Special Report 176 (see Selected Text Reference 1).

Reference 13
SERRATED SOFT-ROCK CUT SLOPES

Richards, Dennis; Ham, David. Arlington, Virginia: U.S. Department of Transportation, Federal Highway Administration, Region 15, R&D Demonstration Projects Division; June 1973; 29 p. (Report # FHWA-RDDP-5-1).

Order from: U.S. Department of Transportation, Federal Highway Administration, Region 15, R&D Demonstration Projects Division, Arlington, Virginia 22201.

In an attempt to control and reduce erosion of soft-rock cut slopes, a simple, economical and expedient method of obtaining quick growth of vegetation on soft-rock cut slopes was developed. This method consisted of serrating (stepping) the cut slopes during the grading process. The serrated slopes consist of a series of steps, 2-4 ft high, with the horizontal dimension or "bench", a function of the slope ratio. The benches provide seed beds for the generation of vegetation. These serrated slopes must be followed by appropriately timed seeding, mulching, and fertilizing to assure good vegetative cover. The serrations gradually disappear, resulting in a straight, well-vegetated slope. A sample guide specification for construction and measurement for stepped slopes is provided, as well as photographs and diagrams of the procedure. Since the slopes need not be scaled after initial excavation, excavation costs are less than those for conventional methods. Excavation quantities that use this technique, are considered to be the same as for non-serrated slopes because the mid-point of the "benches" is set on the staked slope. Normal staking is used to slope-stake the cuts. Technical papers describing successful stepped (serrated) slopes are also included.

Reference 14
DESIGN AND COMPACTION OF COMPACTED SHALE EMBANKMENTS

Bragg, G.H. Jr.; Ziegler, T.W. U.S. Army Engineer Waterways Experiment Station, Soils and Pavements Laboratory. Washington, DC: U.S. Department of Transportation, Federal Highway Administration, Office of Research and Development; September 1975; 245 p. (Report # PB-253121).

Order from: National Technical Information Service, 5285 Port Royal Road, Springfield, Virginia 22161.

The purpose of this three-phase study is to develop design and construction methodologies that will enable shales which have caused settlements and slope failure in highway embankments to be identified and used successfully in future construction. Volume 2 discussed here, considers Phase II. In this phase, techniques for evaluating the stability of existing embankments and remedial treatments for distressed embankments were developed. Information obtained from state and federal agencies and the literature was reviewed. Types and probable causes of distress, evaluation techniques, and remedial treatment measures are discussed. Evaluation techniques recommended are: instrumentation with piezometers, inclinometers, and settlement markers; undisturbed sampling and laboratory testing; in situ testing with borehole devices (Menard pressuremeter and Iowa shear test device); and back analyses of failed slopes. The primary considerations in remedial treatment should be subsurface drainage (mainly horizontal and vertical drains) and surface drainage (mainly paved ditches). Slope flattening, berms, retaining walls, cement grouting, and/or complete reconstruction should be considered in addition to drainage measures when extensive movements and/or shale deterioration have caused large reductions in shear strength. This is the second of two volumes. Volume 1, covering Phase I, is published as FHWA-RD-75-61 subtitle: Survey of Problem Areas and Current Practice.

Reference 15
A STUDY AND ANALYSIS OF TIMBER CRIB RETAINING WALLS

Schuster, Robert L.; Jones, Walter V.; Smart, Steven M.; Sack, Ronald L. Moscow, Idaho: University of Idaho, Department of Civil Engineering; April 1973; 186 p. (Report # PB 221427).

Order from: National Technical Information Service, 5285 Port Royal Road, Springfield, Virginia 22161.

Timber crib retaining walls have been used extensively for many years, particularly in conjunction with logging and mining activities in the Pacific Northwest. Although other kinds of retaining walls have largely replaced the timber crib for many uses, the demand for timber crib walls is great enough to justify the availability of seven basic wall designs. Comparisons were made of various types of retaining walls, including the timber crib, with respect to site adaptation, materials, foundation support and economics. Many in-service timber crib and other types of walls were visually observed to evaluate their performance. Crib-type retaining walls are influenced by a number of soil-structure interactions associated with both the internal stresses and external stability of the retaining wall. In this study, earth pressure inside the timber cribs and the resulting stresses in the crib members and connections have been calculated using the bin pressure theory developed by Janssen. Their resistance to sliding and overturning was determined in the same manner as for other gravity retaining walls. The influence of differential settlement was studied. Because of the myriad of natural occurring soils, four hypothetical groups of soils considered suitable for retaining wall cribfill and backfill were used. In order to predict the relative performance of the timber crib wall designs, each design was completely analyzed for internal stresses and external stability with each of the four hypothetical soil groups. The backfill geometry was also varied from a level to a sloping backfill. Performance of each of the wall designs was then compared and conclusions were drawn about the characteristics and performance of the various wall designs.

Reference 16
REINFORCED EARTH CONSTRUCTION

Walkinshaw, John L. Arlington, Virginia: U.S. Department of Transportation, Federal Highway Administration, Region 15, Demonstration Projects Division; April 1975; 70 p. (Report # PB-247 800/6ST).

Order from: National Technical Information Service, 5285 Port Royal Road, Springfield, Virginia 22161.

This project sought to demonstrate the practicality, cost effectiveness, and aesthetics of reinforced earth structures in highway construction. The reinforced earth concept was developed in France and is a patented process which consists of reinforcing earth with horizontal elements extending from a thin facing (concrete or steel) into a granular backfill to form retaining walls or other types of supporting structures. This report describes the first six retaining structures built on highway projects in the United States, the first bridge abutment and the first foundation slab supporting an embankment in a sinkhole prone geological area. Construction techniques and equipment are discussed as well as costs for each project. Pictures of completed walls illustrate the aesthetics of this concept. This demonstration project has shown that savings in the order of several hundreds of thousands of dollars have been realized on several of the projects described when compared to alternate retaining systems currently available to engineers. The potential for further savings is significant in future highway design. Illustrations of some possible design concepts are presented.

Reference 17
REINFORCED EARTH

Ministry of Equipment. Paris, France: Ministère de l'Equipement, Laboratoire Central des Ponts et Chaussées, April 1976; 23 p. (Note d'Information Technique).

Order from: Ministère de l'Equipement, Laboratoire Central des Ponts et Chaussées, 58 Boulevard Lefebvre, 75732 Paris Cedex 15.

This is a guide to the design and construction of reinforced structures. Although the publication deals only with retaining structures, the rules described may be extended to cover other structures such as foundation rafts and vaults. The theory of reinforced earth is outlined, and technological aspects of soil and reinforcement, service life of reinforced earth projects, and the composition of reinforced earth backfill are considered. Design aspects such as settlements, foundation design, and drainage are covered, as well as details of construction control and supervision. Structures founded on good and poor foundation soil, special situations, and temporary or provisional structures are also discussed. A French edition of this publication is available.

Reference 18
TREATMENT OF SOFT FOUNDATIONS FOR HIGHWAY EMBANKMENTS

Transportation Research Board. Washington, DC: 1975; 25 p. (NCHRP Synthesis of Highway Practice 29).

Order from: Transportation Research Board, Publications Office, 2101 Constitution Avenue, N.W., Washington, DC 20418.

Information is presented on advance planning and preliminary design considerations, subsurface investigation and testing, design techniques, and successful construction alternatives for engineers faced with the problems of highway construction on soft soils. Construction on soft foundations requires extensive investigations and detailed comparative analyses to evaluate possible construction alternatives. Applicable alternatives include (a) elevated structure, (b) embankment fill supported by piles, (c) excavation of soft soils and replacement by suitable fills, (d) subsoil stabilization with or without sand drains, and (e) no treatment, relying instead on especially detailed field investigations and careful design studies to achieve uniform settlement. Where subsoil stabilization involves the use of vertical sand drains, the type of drain influences the design procedures. Conditions of displacement drains are also discussed. Where subsoil consolidation techniques are used, field test sections are desirable to achieve maximum economy. The quality and amount of field inspection are especially important and can be related to postconstruction behavior.

Reference 19
SAWDUST AS LIGHTWEIGHT FILL MATERIAL

Nelson, David S.; Allen, William L. Jr. Washington, DC: U.S. Department of Transportation, Federal Highway Administration, Office of Research and Development; May 1974; 24 p. (Report # PB-233754/AS).

Order from: National Technical Information Service, 5285 Port Royal Road, Springfield, Virginia 22161.

An unstable roadway section was successfully repaired by removing existing high density fill material and replacing it with sawdust. The details of this experiment in slide repair are reported. The project led to the conclusion that sawdust is a very workable fill material and can reduce the driving weight of a potential slide by as much as 71 percent. The fibrous twining of sawdust particles tends to distribute loads in a more lateral direction. Sawdust needs only the compaction of construction equipment driving over it. All indications are that sawdust fills can sustain roadway sections for 15 years or longer. The use of sawdust above the water table should be based on economics and availability, comparing the cost of rehabilitation after the lifetime of the material with alternate solutions. The application of sawdust appears particularly suited to secondary and county roads with unstable soil conditions, where economics often prohibit major slide repair solutions.

Reference 20
CONSTRUCTION OF ROADS ON COMPRESSIBLE SOILS

Organization for Economic Cooperation and Development, Road Research Group, Paris, France; December 1979; 148 p. (Road Research).

Order from: Organization for Economic Cooperation and Development Publications Office, 2 Rue André-Pascal, 75775 Paris Cedex 16.

The report is the result of a study carried out in 1977 and 1978 on construction methods for roads on weak compressible soils, such as clays, silts, and peats. Experts from 16 OECD-member countries participated to review the international state of the art in this field which is gaining in prominence due to the economic, environmental, and energy constraints of modern road design and construction. In addition to reviewing the methods of site investigation, soil testing, stability and settlement analyses, the report gives comprehensive details of methods of treating soft soils to improve the foundation properties, outlines the techniques of instrumentation currently employed, and briefly describes some of the problems that may arise during construction. Several case histories relating to embankment instability are also described. The report ends with a list of conclusions and recommendations, as well as future research needs.

Index

The following index is an alphabetical list of subject terms, names of people, and names of organizations that appear in one or another of the previous parts of this compendium, i.e., in the overview, selected texts, or bibliography. The subject terms listed are those that are most basic to the understanding of the topic of the compendium.

Subject terms that are not proper nouns are shown in lower case. Personal names that are listed generally represent the authors of selected texts and other references given in the bibliography, but they also represent people who are otherwise identified with the compendium subjects. Personal names are listed as surname followed by initials. Organizations listed are those that have produced information on the topic of the compendium and that continue to be a source of information on the topic. For this reason, postal addresses are given for each organization listed.

Numbers that follow a subject term, personal name, or organization name are the page numbers of this compendium on which the term

Indice

El siguiente índice es una lista alfabética del vocablo del tema, nombres de personas, y nombres de organizaciones que aparecen en una u otra de las partes previas de este compendio, es decir, en la vista general, textos seleccionados, o bibliografía. Los vocablos del tema que aparecen en el índice son aquellos que son necesarios para el entendimiento de la materia del compendio.

Los vocablos del tema que no son nombres propios aparecen en letras minúsculas. Los nombres personales que aparecen representan los autores de los textos seleccionados y otras referencias dadas en la bibliografía, pero también pueden representar a personas que de otra manera están conectadas a los temas del compendio. Los nombres personales aparecen con el apellido seguido por las iniciales. Las organizaciones nombradas son las que han producido información sobre la materia del compendio y que siguen siendo fuentes de información sobre la materia. Por esta razón se dan las direcciones postales de cada organización que aparece en el índice.

Los números que siguen a un vocablo del tema, nombre personal, o nombre de organización son los números de página del compendio donde el vocablo o nombre aparecen. Los números romanos se refieren a las páginas en la vista general, los números arábigos se refieren a páginas en los textos seleccionados, y los números de referencia (por ejemplo, Ref. 5) indican referencias en la bibliografía.

Algunos vocablos del tema y nombres de organizaciones están seguidos por la palabra ***see.*** En tales casos los números de página del com-

Index

Cet index se compose d'une liste alphabétique de mots-clés, noms d'auteurs, et noms d'organisations qui paraissent dans une section ou une autre de ce recueil, c'est à dire dans l'exposé, les textes choisis, ou la bibliographie. Les mots-clés sont ceux qui sont le plus élémentaires à la compréhension de ce recueil.

Les mots-clés qui ne sont pas des noms propres sont imprimés en minuscules. Les noms propres cités sont les noms des auteurs des textes choisis ou de textes de référence cités dans la bibliographie, ou alors les noms d'experts en la matière de ce recueil. Le nom de famille est suivi des initiales des prénoms. Les organisations citées sont celles qui ont fait des recherches sur le sujet de ce recueil et qui continueront à être une source de documentation. Les adresses de toutes ces organisations sont incluses.

Le numéro qui suit chaque mot-clé, nom d'auteur, ou nom d'organisation est le numéro de la page où ce nom ou mot-clé parait. Les numéros or name appears. Roman numerals refer to pages in the overview, Arabic numerals refer to pages in the selected texts, and reference numbers (e.g., Ref. 5) refer to references in the bibliography.

Some subject terms and organization names are followed by the word *see.* In such cases, the compendium page numbers should be sought under the alternative term or name that follows the word *see.* Some subject terms and organization names are followed by the words *see also.* In such cases, relevant references should be sought among the page numbers listed under the terms that follow the words *see also.*

The foregoing explanation is illustrated below.

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pendio se encontrarán bajo el término o nombre alternativo que sigue a la palabra *see.* Algunos vocablos del tema y nombres de organizaciones están seguidos por las palabras *see also.* En tales casos las referencias pertinentes se encontrarán entre los números de página indicados bajo los términos que siguen a las palabras *see also.*

La explicación anterior está subsiguientemente ilustrada.

écrits en chiffres romains se rapportent aux pages de l'exposé et les numéros écrits en chiffres arabes se rapportent aux pages des textes choisis. Les numéros de référence (par exemple, Ref. 5) indiquent les numéros des références de la bibliographie.

Certains mots-clés et noms d'organisations sont suivis du terme *see.* Dans ces cas, le numéro des pages du recueil se trouvera après le mot-clé ou le nom d'organisation qui suit le terme *see.* D'autres mots-clés ou noms d'organisations sont suivis des mots *see also.* Dans ce cas, leurs références se trouveront citées après les mots-clés qui suivent la notation *see also.*

Ces explications sont illustrées ci-dessous.

Illustration (from Comp. 1) **Ilustración** (del Comp. 1) **Illustration** (du Recueil 1)

Selected Text page numbers
Números de página en los Textos Seleccionados
Numéros des pages des Textes Choisis

Subject term and *see also* terms
Vocablo del tema y términos *see also* (ver también)
Mot-clé et *see also*

Organization name and address
Nombre y dirección de la organización
Nom et adresse de l'organisation

mountainous terrain (see also degree of curvature; design speed; maximum gradient; radius of curvature; shoulder width): 11, 17, 34, 35, 38, 173, 175, 217, 234, 238

National Association of Australian State Road Authorities(P.O. Box 3141, Brickfield Hill, N.SW. 2000, Australia):
publications, xxiii, Ref. 8, Ref. 9

Overview page numbers and reference number
Número de página en la Vista General y números de referencia
Numéro des pages de l'Expose et numéros des références

no-passing markings and signs: 31, 95, 132

non-passing sight distance, see stopping sight distance

Odier, L.: Ref.3

Oglesby, C.H.: 231, 233, 234, 239, 240, 241, 242, Ref.14

Subject term and *see* term
Vocablo del tema y término *see (ver)*
Mot-clé et *see*

Selected Text page numbers and reference number
Números de página en los Textos Seleccionados y número de referencia
Numéros des pages des Textes Choisis et numéros des références

Personal names
Nombres personales
Noms propres

Project Publications

The publications listed below have been produced in the Transportation Technology Support for Developing Countries project and may be ordered from TRB postpaid at the prices shown.

Publicaciones del Proyecto

Las publicaciones citadas abajo fueron publicadas en el proyecto Apoyo de Tecnología de Transporte para Países en Desarrollo y pueden ser pedidas franco de porte al TRB a los precios indicados para cada publicación.

Publications du Project

Les publications citées ci-dessous ont été publiés par le projet sur la Technologie des Transports pour les Pays en Voie de Développement et peuvent être commandées en port-payé au TRB. Les prix sont indiqués pour chaque publication.

Transportation Research Board, National Research Council, 2101 Constitution Avenue, N.W., Washington, DC 20418 USA

Compendium 1:
Geometric Design Standards for Low-Volume Roads. $12.00
Normas de diseño geométrico para caminos de bajo volumen.
Normes de dimensionnement géométrique pour routes à faible capacité.

Compendium 2:
Drainage and Geological Considerations in Highway Location. $12.00
Consideraciones de drenaje y geológicas en la ubicación de carreteras.
Considérations sur les facteurs de drainage et de géologie qui influencent le choix de l'emplacement d'une route.

Compendium 3:
Small Drainage Structures. $12.00
Pequeñas estructuras de drenaje.
Petits ouvrages de drainage.

Compendium 4:
Low-Cost Water Crossings. $9.00
Travesías de agua de bajo costo.
Ouvrages de franchissement d'eau économiques.

Compendium 5:
Roadside Drainage. $9.00
Drenaje del borde de la carretera.
Drainage des bas-côtés de la route.

Compendium 6:
Investigation and Development of Materials Resources. $10.50
Investigación y desarrollo de recursos de materiales.
Investigation et développement des gisements de matériaux routiers.

Compendium 7:
Road Gravels. $9.00
Gravas.
Les graviers.

Compendium 8:
Chemical Soil Stabilization. $10.50
Estabilización química de suelos.
La stabilisation chimique des sols.

Compendium 9:
Control of Erosion. $12.00
Control de erosión.
Contrôle de l'érosion.

Compendium 10:
Compaction of Roadway Soils. $10.50
Compactación de suelos viales.
Le compactage des sols routiers.

Compendium 11:
Decision Methodology for Maintenance and Upgrading: Costs, Traffic, and Benefits. $10.50
Metodología de decisiones en la conservación y mejora: costos, tránsito y beneficios.
Méthodologie de prise de decision en matière d'entretien et d'amelioration: coûts, traffic et advantages.

Compendium 12:
Surface Treatment. $12.00
Tratamiento de superficie.
Revêtements superficiels.

Compendium 13:
Slopes: Analyses and Stabilization. $10.50
Taludes: Análisis y estabilizacíon.
Talus: Analyses et stabilisation.

Synthesis 1:
Maintenance of Unpaved Roads. English Edition. $6.00
Conservación de caminos sin pavimentar. Versión en español. $6.00
L'entretien des routes non revêtues. Version française. $6.00

Synthesis 2:
Stage Construction. English Edition. $6.00
Construcción por etapas. Versión en español. $6.00
Aménagement progressif. Version française. $6.00