DISSERTATION

GULLY INTRUSION INTO RECLAIMED SLOPES:

A LONG-TERM TIME AVERAGED CALCULATION PROCEDURE

Submitted by

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In partial fulfillment of the requirements

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ABSTRACT OF DISSERTATION

GULLY INTRUSION INTO RECLAIMED SLOPES:
A LONG-TERM TIME AVERAGED CALCULATION PROCEDURE

Increased environmental concern throughout the United States has renewed the importance of mined land reclamation and disposal of wastes associated with the excavation and processing of natural resources such as coal, uranium and other minerals. The Surface Mining Control and Reclamation Act of 1977, Public Law 95-87, requires mining companies to restore the approximate original contour of the land in a manner that prevents slides, erosion, and water pollution. The Uranium Mill Tailings Radiation Control Act of 1978, Public Law 95-604, outlines measures to stabilize radioactive byproducts and restoration of mine sites from which these materials were removed. The restoration of mined land and long-term stabilization of waste disposal areas is of priority to protect public health, safety and conserve other natural resources, such as soil and water. Current regulations (40 CFR 192) require the waste to remain undisturbed, usually for periods of 200 to 1,000 years.

The impoundments constructed to encapsulate the waste materials must resist the natural erosive processes to prevent exposure and release of the waste. One erosive process that has been difficult to predict is gully intrusion. This study was conducted to analyze the gully erosion processes and determine its effect on long-term embankment stability and estimate the potential impacts on waste stabilization. The investigation was conducted in four distinct phases: (1) literature search and background information collection; (2)

performance of a laboratory simulation study; (3) field data collection; and (4) data compilation, analysis, and synthesis.

A comprehensive procedure has been presented for estimating the magnitude and location of a potential gully intrusion into a soil covered, waste impoundment. The estimation procedure requires that the user obtain information pertaining to the regional hydrology, soil characteristics, proposed impoundment geometry, and design life. It is noted that while a limited set of field data was available for synthesis into the prediction equations, the procedure presented is a first step into the determination of the magnitude and location of gullying into sloped surfaces.

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Finally, I would like to thank my wife Tonya, not only for putting up with me while I finished this research, but for providing the love and tenderness that makes my life special.

DEDICATION

For my Grandpap and Grandfather. You are dearly missed.

Harry Norton Hill

and

Kirby Fairfield Thornton

"... and if I search. I know I'll find

a part of them in me.

and through it all I'll always have

a guide on land and sea."

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CHAPTER 1

INTRODUCTION

Increased environmental concern throughout the United States has renewed the importance of mined land reclamation and disposal of wastes associated with the excavation and processing of natural resources such as coal, uranium and other minerals. The Surface Mining Control and Reclamation Act of 1977, Public Law 95-87, requires mining companies to restore the approximate original contour of the land in a manner that prevents slides, erosion, and water pollution. The Uranium Mill Tailings Radiation Control Act of 1978, Public Law 95-604, outlines measures to stabilize radioactive byproducts and restoration of mine sites from which these materials were removed. The restoration of mined land and long-term stabilization of waste disposal areas is of priority to protect public health, safety and conserve other natural resources, such as soil and water. Current regulations (40 CFR 192) require the waste to remain undisturbed, usually for periods of 200 to 1,000 years.

Current stabilization methods (10 CFR 40, Appendix A) for waste disposal recommend that the waste material be placed below the ground surface. Below ground disposal may not be feasible if ground water resources are at risk or if geologic conditions, such as near surface bedrock, prevent cost effective burial. When below ground burial is not possible, an earthen cap or cover is placed over the waste material. Above ground impoundments can have embankment and cover slopes ranging from 5 to 30%. The regulations require a "self-sustaining" vegetative or rock cover to minimize erosion of the impoundments. Where large areas, greater than 20 acres, of erosion protection are required, the use of riprap, synthetic fabrics, and artificial barriers can be prohibitive due to costs. Whether vegetated, rocked or unprotected, waste material can be susceptible to exposure resulting from gully erosion. Figure 1.1 illustrates a gully that may intrude into a typical earthen tailings impoundment. The design objectives for protective covers are four fold: 1) to prevent tailings exposure due to erosion, 2) to insure long-term stability, 3) to provide for minimal maintenance requirements, and 4) to accomplish the design objectives in an efficient and effective manner.

While waste impoundments are required by current laws and regulations to be stable and "self-sustaining," the design engineer has the added burden of public health and safety. Gully intrusion into a waste impoundment poses several threats to the public. First and foremost is the potential for the waste cell to be breached and the hazardous material exposed to the environment. Even if a gully does not breach the buried waste cell, the depth and location of the gully could provide insufficient cover to the cell. In addition, the sediment transported from the embankment during the process of gully migration could be detrimental to the downstream ecosystem.

The goal of this study was to analyze the gully erosion processes and determine these effects on long-term embankment stability and potential impacts on waste stabilization. A series of project objectives were identified as essential steps in meeting the project goals. They are as follows:

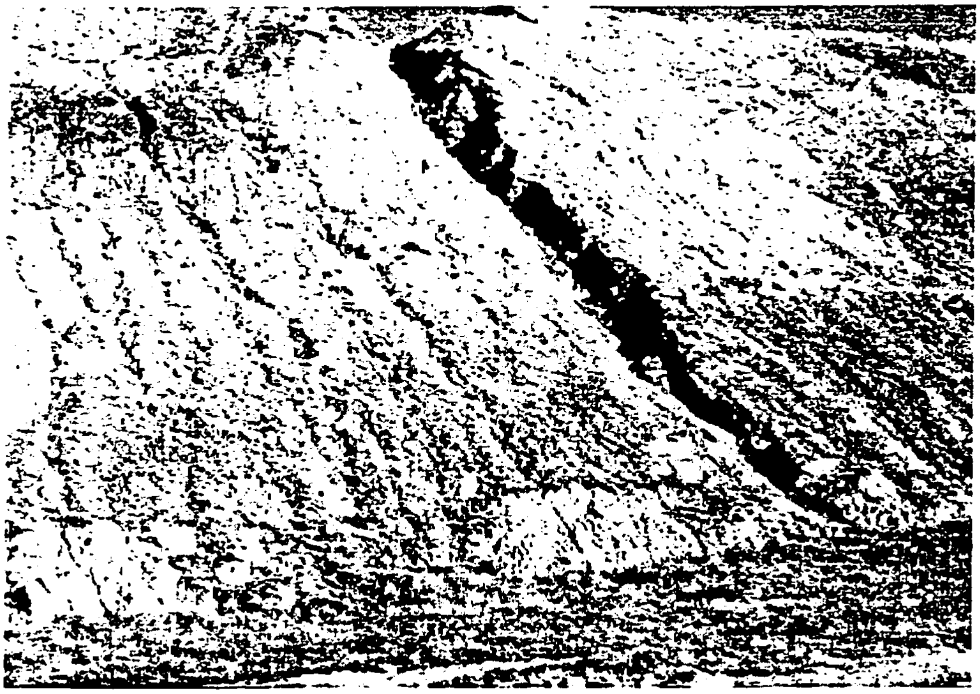

Figure 1.1 Typical Gully on Waste Impoundment

- A review of the available literature will serve to define the current state of practice in estimating the effects of gully formation, as well as provide a knowledge and database. Studies that incorporate field and flume data on the formation, migration, and effects of gullies will be identified. Of particular interest will be studies that incorporate field data collected from actual reclaimed lands.
- An analysis of the erosive processes present in the formation and migration of a gully will provide essential insight necessary to achieve the project goal. Gully processes will be examined to insure that appropriate variables are considered and incorporated into the design procedure.
- A database will be formed from the available literature that includes a quantitative description of the prominent variables associated with gully formation. It is imperative that the studies included in the database contain the same variables.
- Impoundment geometries typically implemented in the reclamation of mined or disturbed lands will be identified. This information will be used in the selection of appropriate studies to make up the database. While there is the intent to be able to extrapolate the results of this study's other applications, the goal of the research is to provide a methodology and design procedure capable of estimating the effects of gully intrusion to the reclamation industry.
- An analysis will be conducted to develop a set of empirical relationships that can be used to predict the location, depth, bottom slope, and width of a gully given a specific site geometry.

- A procedure that will enable an engineer to design an above grade waste impoundment against the long-term, time averaged effects of gully intrusion will be developed. A step by step methodology will be presented that incorporates the various geometric, hydrologic, hydraulic, and temporal conditions typically associated with mine reclamation.

By meeting the outlined objectives, and realizing the goal of the project, design engineers will have a tool to predict the long-term, time averaged (200 to 1,000 years) stability of an embankment from potential gully incision. By using the design procedure, developed through this research, an engineer will be able to evaluate the stability of a proposed waste impoundment and examine the potential consequences of gully incision into the embankment.

CHAPTER 2

LITERATURE REVIEW

2.1 GULLY EROSION

A literature review was conducted in an effort to understand and quantify the mechanisms and variables common to the different forms of gully erosion. While the purpose of this study is to present a long-term, time averaged approach to estimating the effects of gully erosion, a review of the definitions and features common to gullies were examined.

2.1.1 Definition of a Gully

By definition of the Soil Conservation Service (SCS) (1966), Hudson (1985) and the Glossary of Geology (Bates and Jackson, 1980), a gully is any channel with a width and/or depth that makes tillage by normal farm equipment impossible. The Glossary of Geology (Bates and Jackson, 1980) further defines a gully as "a long, narrow hollow or channel worn in earth or unconsolidated material by running water." The Glossary of Geology also defines gully erosion as "erosion of soil or soft rock material by running water that forms distinct, narrow channels that are larger and deeper than rills and that usually carry water only during or immediately after heavy rains or following the melting of snow or ice."

2.1.2 Characteristics of Gullies

Imeson and Kwaad (1980), Foster (1984), Hudson (1985), Ebisemiju (1989) and Bocco (1991) indicated that gullies exhibit several common general characteristics. Gullies commonly, but not always, form as a result of concentrated flows due directly or indirectly to the work of man. Gullies often lie above the water table. Gully side walls are typically vertical or near vertical depending on the specific erosional processes at work, soil characteristics, and the development stage of the system. Most gullies, especially active ones, lack vegetation on the side walls and bottoms. Gully growth is typically rapid and progressive, but can be cyclic, steady or spasmodic. The eroded soil material is typically unconsolidated slope deposits, weak shales and deeply weathered. Gully flow is typically ephemeral in nature and only partially fills the channel even at high stages.

2.1.3 Classification of Gullies

Several authors have classified gullies based on form, shape and morphological processes, however a universal classification system does not exist for gullies. One of the classic studies of gullying was conducted by Ireland *et al.* (1939). Ireland stated that the characteristic gully forms were a result of physical and land use factors affecting the drainage basin and thus classified gullies according to their outline and form. Ireland observed six common forms of gullies and defined them as: linear, bulbous, trellis, parallel, dendritic, and compound.

In 1956, Leopold and Miller were the first to designate gullies as either continuous or discontinuous. A discontinuous gully is characterized by the presence of a sharp headcut at the mouth of the gully. By definition, a discontinuous gully increases rapidly in depth downslope of the headcut, but decreases in depth near the downstream end where it intersects the original slope and forms an alluvial fan. Heede (1976) observed discontinuous gullies forming on a mountain slope and thought their initiation was a result of a break in the original slope gradient. It was thought that groups of discontinuous gullies are the precursors to the formation of a continuous gully system.

Heede reported, in 1976, that a continuous gully typically begins with finger-like extensions into the headwater area. In addition, the continuous gully gains depth rapidly downstream and maintains that depth nearly to the outlet. Just before the mouth of the gully, the depth decreases rapidly until the outlet is reached. Leopold and Miller (1956) and Heede (1976) stated that continuous gullies can form from the combining of discontinuous gullies. Although the definitions of discontinuous and continuous gullies provide a classification system, the actual classification can be difficult to apply in the field where distinctions between the two types can be varied and vague.

A classification system based on the shaping processes and morphological characteristics of gullies was proposed by Imeson and Kwaad (1980). Four types of gullies were defined based on the cross sectional shape of the gully, the location in the landscape, principle runoff source, and the soil matrix. Type 1 gullies were defined as having a V-shaped cross section where the principle runoff source is overland flow. Typically, Type 1 gullies occur in all landscape positions, except valley bottoms, where the underlying soil is a relatively resistant, deep soil. Gullies classified as Type 2 have similar properties as Type 1 gullies with the exception being a U-shaped cross section. Gullies with a U-shaped cross section that are in areas of the landscape where subsurface flow, mainly in the form of piping, predominates are classified as Type 3. Type 4 classification is reserved for gullies found exclusively in the valley bottom where flow is derived mainly from tributary gullies and overland flow.

2.1.4 Causes of Gully Erosion

Numerous causes of gully erosion were described by Elliott (1990) and Leopold *et al.* (1964). Both papers reported that the primary cause of gully erosion was a combination of an increase in surface runoff and the concentration of surface flow. Natural and man induced influences have been observed to influence the quantity and concentration of surface flow.

Leopold *et al.* (1964) report that natural causes are often the result of climate changes. Drying trends and droughts can leave the landscape ripe for increased gully erosion when the rains return. In addition, the natural lowering of the base level in local streams has been reported to increase gully activity within the drainage basin.

Local disturbances due to human influences can also cause channelization and generally increase surface runoff. The influx of cattle grazing has also been mentioned as a possible cause of accelerated gullying in the western United States during the 1800s. Cattle grazing can increase runoff by removing vegetation and reducing soil infiltration rates due to hoof compaction (Clary, 1996). Road, mine and railway construction can contribute to gullying. Roads provide large impermeable surface areas. Drain structures along roads, mines and railroads tend to concentrate flows and if improperly designed can promote gully erosion.

Surface mining can substantially alter pre-disturbance hydrologic and geologic conditions. Elliott (1990) noted that changes in soil properties, soil horizons, and geologic structural controls can alter the surface runoff characteristics of a mine site. The loss of vegetation due to agricultural development, mining or even natural drought can promote gully growth. Changing the geologic controls, vegetative cover, and geomorphology of a drainage basin during development can affect the quantity and frequency distribution of runoff thereby increasing the potential for gully erosion.

2.2 GULLY EROSION PROCESSES

Ireland *et al.* (1939) produced "Principles of Gully Erosion in South Carolina," one of the most complete works describing the processes and mechanisms of gully erosion. Five processes contributing to gully enlargement were identified in the literature: stream power, downcutting, headcutting, bank instability, and piping. The literature also indicated that recognition of gully evolution and growth stages were essential to properly analyzing gully erosion.

2.2.1 Stream Power

The importance of the stream power concept to gully erosion has been investigated by several authors, Piest *et al.* (1972, 1975), Bradley (1980), Elliott (1990), and Hanson *et al.* (1997). Stream power was defined by Bagnold (1966) as "the time rate of energy supply to a unit length of a stream." Bagnold (1966) defined the average stream power per unit channel bed area (ω) as:

$$\omega = \frac{\rho g Q S}{W} \tag{2.1}$$

where: ρ = the fluid density;

g = the acceleration due to gravity;

Q = the discharge rate;

S = the energy slope; and

W = the flow width.

Bradley (1980) and Elliott (1990) were able to estimate a gully stream power parameter using field data and several assumptions. The stream power parameter was used to estimate threshold values which when exceeded, would initiate gully erosion. The assumptions employed included substituting the bed slope for the energy slope (S) and the valley floor width for the flow width (W). The discharge (Q), seldom known in the field, was estimated by the contributing drainage basin area (Burkham, 1966).

Piest *et al.* (1972, 1975) determined that tractive force and stream power were not the primary processes causing gully erosion in the loess soils of western Iowa. The study observed mass wasting of walls and headcutting as the dominant growth process. These observations were confirmed by monitoring discharge of gully sediments during storm events over the course of several years. Stream power and tractive force probably are the dominate processes only during initiation of gully growth and during early downcutting stages.

Hanson *et al.* (1997) reported on 41 flume experiments designed to relate the rate of headcut migration to stream power dissipation, where stream power was defined as:

$$\dot{E} = q \gamma H \tag{2.2}$$

where: $\dot{E}$ = the flow dissipation rate per unit width of headcut (Kw/m);

q = the unit width discharge (m^2/sec);

γ = the unit weight of water (N/m^3); and

H = the energy head dissipated through headcut (m).

The analysis presented by Hanson *et al.* (1997) found that headcut migration rates could be characterized by a linear regression of headcut movement over time. However, it was found that stream power dissipation through the headcut provided no real correlation to headcut migration. Results indicates that headcut migration rates were significantly correlated to the following soil parameters: dry unit weight, moisture content, jet index, unconfined compressive strength, and failure strain.

2.2.2 Downcutting

Downcutting is defined by the Glossary of Geology (Bates and Jackson, 1980) as "stream erosion in which the cutting is directed in a downward direction (as opposed to lateral erosion)." As noted by Ireland *et al.* (1939), small concentrations of flow or extraordinary large rainfall events can initiate downcutting and the process is only active during runoff events. Ireland *et al.* (1939) reported that downcutting is primarily a function of the surrounding soil, gully slope, and volume of runoff.

2.2.3 Headcutting

Brush and Wolman (1960) performed a laboratory study on knickpoint migration in cohesive soils, determining that knickpoint migration is a function of flow rate, eroded material size and the fall distance between the top and bottom of the knick. Knickpoints commonly have non-vertical slopes but can migrate upstream with a vertical face depending on specific conditions of the channel.

Stein and Julien (1993) defined a headcut as the most upstream knickpoint within the gully where the original terrain slope transitions into the gully bottom slope. Ireland *et al.* (1939) reported that the formation and growth of a distinctive headcut signifies the beginning of the rapid enlargement stage of gullying. Several mechanisms of headcutting such as: lip scour, plunge-pool undercutting, caving and seepage were identified by Ireland *et al.* (1939). The headcutting mechanisms can be present individually or in any combination. In an effort to classify gully headcuts, Ireland *et al.* (1939) devised a system to group gully heads according to their vertical profile and rim outline. Figure 2.1 presents a schematic of the classification system.

Four gully head profiles: inclined, vertical, cave, and vegetated were identified and are labeled in Figure 2.1. Inclined gully heads are characteristically low heads occurring entirely within one soil horizon. While vertical heads are observed regularly in open channel flow, they are not typically found in gullies. Vertical heads may be within one soil horizon or may form in a material of uniform resistance. The most common type of head in deep gullies where soil horizons are of varying degrees of resistance is the cave. Cave heads can be formed by seepage and/or plunge pool action. Vegetated heads are formed when an overhanging root mass or sod plot forms a spout and prevents water from impacting the bank.

Four gully rim outlines: pointed, rounded, notched, and digitate are also schematically defined in Figure 2.1. Pointed gully rims are characteristic of shallow channels that deepen and broaden gradually and uniformly downstream of the pointed head.

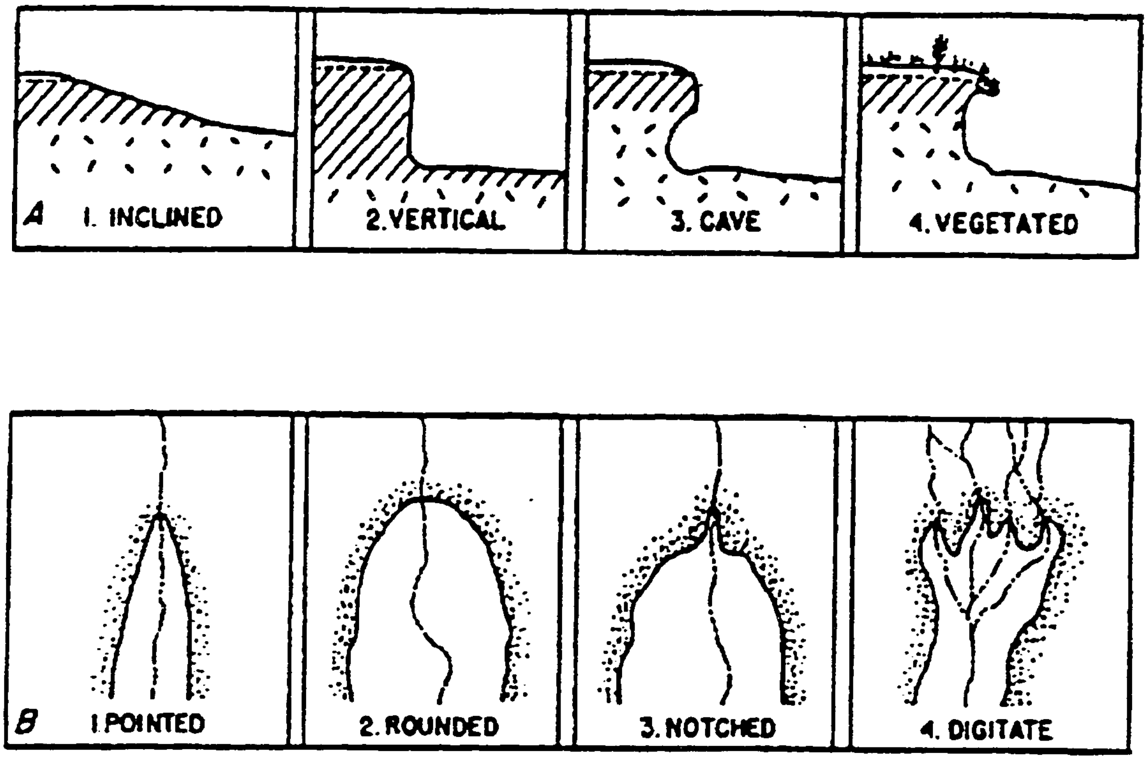

Figure 2.1 Plan and Profile of Characteristic Gully Heads (Ireland *et al.*, 1939)

Rounded gully rims exhibit a semicircular shape typically with near vertical walls that produces a symmetrical gully downstream of the head. In situations where the gully rim has formed in a resistant soil horizon and the channel has cut down through a weaker material, a notched rim is formed. Shaped similar to the rounded rim, a notched gully rim exhibits a sharp break, or notch, within the semicircle. Digitate gully rims form in the shape fingers on a hand with multiple heads extending upstream.

The SCS (1966) reported that factors such as: drainage area above the headcut, number of precipitation events, soil characteristics, gully slope, and ground water levels can affect the rate of gully head advancement. Headcut migration is very similar in process to the factors contributing to bank instability.

2.2.4 Bank Instability

Processes such as freezing and thawing, caving, seepage, wetting and drying, and slumping have been identified as affecting bank instability (Ireland *et al.*, 1939). Schumm (1960) reported that the shape of a stable alluvial channel is a function of the bed and bank material and was able to predict the width to depth ratio of a channel cross section based on the percentage of silt and clay in the soil. In 1973, Bradford examined gully head and wall stability. He researched failure types and material properties and reported that stability was affected by: internal friction angle of the soil, soil cohesion, infiltration rate, and the elevation of the water table. In 1977, Bradford then attempted to apply Bishop's limiting equilibrium slope stability method to gully wall stability. While unsuccessful, Bradford reported that stability was not affected by: soil bulk density, slope height or the presence of side wall tension cracks. Heede (1976) reported that wall collapse of gullies with high clay content was mainly due to a repeated wetting and drying cycle, and that the wetting process typically occurred during precipitation events that did not cause gully flow.

2.2.5 Piping

Piping, defined by the Glossary of Geology (Bates and Jackson, 1980) as "erosion by percolating water in a layer of subsoil, resulting in caving and in the formation of narrow conduits, tunnels, or 'pipes' through which soluble or granular soil material is removed," was determined to be more important to gullies forming in arid climates than in humid climates (Brown, 1962; Parker and Jenne, 1967). Leopold *et al.* (1964) attributed piping to accelerated headcut migration, and bank and wall instability. Heede (1970) indicated that piping is a function of soil sodium and gypsum content, soil texture, montmorillonite clay content of the soil, and hydraulic head. Haggerty (1991) observed piping and sapping in alluvial stream banks where soil layers provided zones for conduit flow and a source of recharge was available.

2.2.6 Stages of Gully Development

Ireland *et al.* (1939) presented a four stage growth sequence for gullies. Stage 1 is characterized by downward scour and Stage 2 is characterized by headward cutting and rapid enlargement. The main features of Stages 1 and 2 were described in Sections 2.2.2 and 2.2.3, respectively. Stage 3, the healing stage, is started by diversion of the runoff source or areal diminishing of the contributing basin due to headcut advancement. The sensitive Stage 3 period, can revert back to Stage 2 due to a local disturbance. Stage 4, the stabilization stage, is characterized by the channel slope obtaining semi-permanent base level, bank slopes near the soil angle of repose and vegetation in sufficient quantities to anchor the soil. As with Stage 3, Stage 4 can revert to any previous stage if the state of equilibrium is disturbed.

Bradford *et al.* (1973) presented a three phase growth sequence. Gully head and bank failure are followed by clean out of the debris by stream flow and then degradation of the channel. This three phase sequence assumed the gully already exists. That is, formation was beyond an early growth period, such as rilling, where vertical degradation and channel clean out predominate.

Heede (1974, 1976) divided gully evolution into youthful and mature periods. The youthful stage was characterized by: discontinuous gullies, small order drainages, and longitudinal profiles with weak concavities. The dominant characteristic of a mature gully was some type of dynamic equilibrium. This equilibrium condition was not a permanent balance of forces but a state capable of readjusting fairly quickly to induced fluxes on the gully system. The mature gully should also exhibit increased vegetation cover. Heede stated that a mature gully may return to the youthful stage due to any one of the following: changes in land use, changes in climate, uplift of the drainage area or lowering of the downstream base level. Gully growth does not truly follow any ordered steps. Growth can be episodic and after rainfall events a gully can be left in a stable or very unstable condition.

2.3 GULLY EROSION MODELS

Numerous gully erosion models are available in the literature. The models cover many aspects of gully growth and utilize a variety of variables to represent the gully erosion processes. All the models presented have empirically determined coefficients and are site specific in their development.

2.3.1 SCS Flood Prevention Program (U.S. Department of Agriculture (USDA, 1943))

The SCS (USDA, 1943) studied 29 gullies along the Little Sioux River in western Iowa to determine economic feasibility of gully control measures for a 50-year design period. The SCS found that the change in gully area is not a linear process and that past growth rates could not be linearly extrapolated in order to predict future growth rates. The study reported that development of the Little Sioux gully system would follow a general s-curve growth equation of the form:

$$\text{Cumulative effect} = \frac{1}{1 + Ce^{-kt}} \tag{2.3}$$

where: C, k = constants; and

t = the age of the system.

After evaluating the constants for the Little Sioux gullies. Equation (2.3) was used to predict future amounts of basin damage.

$$\begin{matrix}\text{Percent of contributing} \\ \text{area damaged}\end{matrix} = \frac{100}{1 + 1000e^{-.07t}} \tag{2.4}$$

2.3.2 Beer and Johnson (1963)

Beer and Johnson (1963) compared, during an investigation of areal gully expansion in western Iowa, a linear model of the form:

$$\text{Cumulative Effect} = b + c_2X_2 + c_3X_3 + \ldots + c_nX_n \quad (2.5)$$

with a logarithmic model of the form:

$$\text{Cumulative Effect} = b * X_2^{c_2} * X_3^{c_3} * \ldots * X_n^{c_n} * e^{c_{n+1}X_{n+1}} \quad (2.6)$$

In Equations (2.5) and (2.6), b and c_2 through c_{n+1} are regressed constants, and X_2 through X_n are various gully system parameters. The X_{n+1} parameter in Equation (2.6) is the deviation of precipitation from normal in inches. Beer and Johnson determined that the logarithmic model was a better predictor of change in areal gully growth because the average deviations from observed values for this model were smaller than those of the linear model. In addition, the linear model was capable of predicting negative values whereas the logarithmic model was forced through zero eliminating the negative predictions. For the loessial area of western Iowa, in which the study was conducted, the recommended relationship was presented as Equation (2.7):

$$X_1 = 0.01X_4^{0.0982}\, X_6^{-0.0440}\, X_8^{0.7954}\, X_{14}^{-0.2473}\, e^{-0.0360X_3} \quad (2.7)$$

where: X_1 = the change in gully surface area (acres);

X_3 = the deviation of precipitation from normal (in.);

X_4 = the index of surface runoff (in.);

X_6 = the terraced area of watershed (acres);

X_8 = the gully length at beginning of period (ft); and

X_{14} = the length from end of gully to watershed divide (ft).

The authors noted that some variables in Equation (2.7) were not universally applicable. For instance X_1 is not always 0 when $X_6 = 0$.

2.3.3 Thompson (1964)

Thompson (1964) based his model of gully head advancement on 210 increments of gully growth in seven different gullying areas of the United States. He utilized a logarithmic multiple regression model incorporating five watershed characteristics as independent variables. The dependent variable was gully head advancement (R) in feet. The independent variables were drainage area above the gully head (A) in acres, approach slope of channel above the gully head (S) in percent, gully depth (D) in feet, summation of rainfall from 24-hour rains equal to or greater than 0.5 in. (P) in inches, and soil factor (E) for the soil through which the gully is advancing (percent of material 0.005 mm or smaller). The author performed forward regression with the independent variables and recommended Equation (2.8) to describe gully head advancement for the study.

$$R = 0.15 A^{0.49} S^{0.14} P^{0.74} E^{1.00} \tag{2.8}$$

2.3.4 Seginer (1966)

Seginer (1966) used a logarithmic model to predict long-term (hundreds of years) gully head advancement in southern Israel. He assumed a non-changing watershed, one in which vegetation cover and basin area are relatively constant. By assuming a constant mean level of hydrologic activity, he postulated that long-term rain quantities and time could be linearly correlated. Over the long term, he assumed the affects from precipitation would dominate all other independent variables. In the long term, precipitation is the dominant independent variable and all other independent variables would only affect the gullying process on a short-term (a few years) basis. Seginer was not able to utilize the variables of precipitation and soil type in his analysis due to a lack of variance and correlation. He did determine that the watershed geometric properties of: area, length, and maximum elevation difference were important in describing gully head advancement. Using area (A) as the dominant indicator of gully head advancement, he developed the relation:

$$E = c_1 A^{0.50} \tag{2.9}$$

where: E = the average annual advancement in meters; and

c_1 = a coefficient determined for the particular study location.

Seginer also developed a long-term advancement prediction procedure based on Equation (2.9).

2.3.5 United States SCS (1966)

Using data from Thompson's (1964) study, the SCS (1966) developed a procedure to solve field design problems involving gully erosion. The recommended simplified equation is:

$$R_f = R_p \, A^{.46} P^{.20} \tag{2.10}$$

where: R_f = the computed future average annual rate of gully head advance for a given reach, in feet per year;

R_p = the past average annual rate of gully head advance, in feet per year;

A = the ratio of the average drainage area of a given upstream reach to the average drainage area of the reach through which the gully has moved; and

P = the ratio of the expected long-term average annual inches of rain from 24-hour rainfalls of 0.5 inch or greater to the average annual inches of rain from 24-hour rainfall of 0.5 inch or greater for the period, if less than 10 years, in which the gully head has moved.

2.3.6 Falk (1985)

The objective of Falk's (1985) study was to predict the maximum incision depth (D_{max}) and the location of D_{max} in a gully on reclaimed mine tailings slopes. Falk measured and sampled characteristics from 16 gullies on spoil piles in northern Colorado and Wyoming. Falk made several assumptions including: the rate of gully growth decreases with time and is proportional to the initial slope angle, the reclaimed slope is homogenous, the contributing drainage area of a gully remains constant, only storms equal to or greater than 0.5 inches cause gully growth, pile settlement effects were negligible, vegetative cover was constant, and location and elevation of the toe remained constant and was not affected by rill or gully erosion. Falk found that a dimensional analysis of the collected variables was unsuccessful in producing a parameter with significant correlation in describing field observations. Falk indicated that the rate of gully incision is a function of drainage basin area, number of rain events that cause runoff, initial slope angle, and average particle size (D_{50}). The top width of the gully was also found to be a function of D_{max} and the soil coefficient of uniformity (C_u). The location of D_{max} was determined to be a function of gully bottom slope and the soil coefficient of uniformity (C_u).

2.3.7 Pauley (1993)

A large scale laboratory study was conducted by Pauley in an effort to quantify the size and location of a gully forming on a sloped surface. Two facilities were constructed for the experiments. Seven tests were conducted in an outdoor flume at near prototype conditions, and eight tests were conducted in an indoor flume at a reduced scale. The focus of the experiment was to analyze combinations of soil types, initial slope angles, and soil compaction on gully formation.

Pauley (1993) investigated two empirically derived prediction models, a linear and nonlinear multiple regression. Due to the nonlinear growth phases proposed by the SCS (1966), two "S" curves were developed to relate maximum depth of gully incision to pile height and cumulative runoff volume. Equations (2.11) and (2.12) present the prediction equations for a most probable and worst case scenarios, respectively.

$$\frac{D_{max}}{H} = \frac{1}{1.22 + (0.0002\,V_R)^{-0.5}} \qquad (2.11)$$

$$\frac{D_{max}}{H} = \frac{1}{1.2 + (0.012\,V_R)^{-0.7}} \qquad (2.12)$$

where: D_{max} = the maximum depth of gully incision (m);

H = the embankment height (m); and

V_r = the total volume of runoff tributary to the embankment (m^3).

2.3.8 Abt *et al.* (1994)

Abt *et al.* (1994) utilized results of a large scale laboratory study, Pauley (1993), and a field investigation of gullying on actual reclaimed impoundments to develop a methodology for predicting the maximum depth of gully incision into a reclaimed embankment. The analysis focused on relating the maximum depth of gully incision, top width of the gully, the location of the maximum incision to volume of tributary runoff, pile height, and soil composition.

A series of three envelope curves were generated, based on soil composition, to quantify the maximum gully depth caused by incision. The relationships determined by Abt *et al.* (1994) are as follow:

Clay content less than 15%:

$$\frac{D_{max}}{H} = \frac{1}{1.7 + (0.0194\,V_R)^{-0.9}} \tag{2.13}$$

Clay content greater than 15% but less than 50%:

$$\frac{D_{max}}{H} = \frac{1}{1.9 + (0.0053\,V_R)^{-0.95}} \tag{2.14}$$

Clay content greater than 50%:

$$\frac{D_{max}}{H} = \frac{1}{2.15 + (0.0021\,V_R)^{-1.3}} \tag{2.15}$$

where: D_{max} = the maximum depth of gully incision (m);

H = the embankment height (m); and

V_r = the total volume of runoff tributary to the embankment (m^3).

In addition, Abt *et al.* (1994) presented an envelope relation for estimating the width of a gully, at the location of maximum incision, on a reclaimed slope. Maximum depth of incision was plotted against width at the location of maximum incision and a predictive and envelope relation developed. The predictive equations for the power regression and envelope relations are presented in Equations (2.16) and (2.17), respectively.

$$D_{max} = 0.618\,(W)^{0.89} \tag{2.16}$$

$$D_{max} = 1.58\,(W)^{0.80} \tag{2.17}$$

where: D_{max} = the maximum depth of gully incision (m); and

W = the gully width at location of maximum depth of incision.

2.4 SUMMARY

A review of the literature has indicated that the evolutionary development of gullies and the mechanics of the gulling process are well documented. In addition, there are several methods available to predict the rate of headcut advancement and the percentage of land consumed by gully erosion. Without exception, the methods presented in Section 2.3 require that the depth of the gully at a given point in time be known. In addition, the available equations for predicting gully growth are site specific and only incorporate the variables relevant to the process (i.e., areal change or headcut advancement) of concern.

The goal of this research is to develop a long-term, time averaged methodology and calculation procedure allowing the design engineer to estimate the location and depth of the maximum point of gully incision on an embankment.

Many variables have been identified as factors in the gully erosion process. The important variables can be grouped into three broad categories: hydrologic variables (precipitation and evaporation), geometric variables (basin dimensions, terrain and gully slopes, terraced areas), and geotechnical and/or variables (soil parameters, geologic conditions and vegetative coverage). However as Heede reported in 1976, "no controlled studies of the individual components responsible for the gullying process have been made." Data sets incorporating variables describing soil properties, hydrologic events and site geometry are necessary to investigate the highly nonlinear relation between the variables and a long-term time averaged approach to gully estimation.

CHAPTER 3

DATABASE ASSEMBLY

A review of the literature indicates that while the evolution of gully processes are well documented, a method to examine a time averaged approach to gully formation does not exist. With a design requirement of impoundment stability for periods of 200 to 1,000 years imposed by Congress, engineers in the reclamation industry are tasked with looking beyond an event based approach to waste impoundment design. From a practical standpoint, the designer must be able to estimate the maximum depth a gully could reach, its location along the embankment, and then design the impoundment to provide an acceptable level of safety.

Gully migration equations found in the literature identify many important variables in describing the gullying process. These variables can be grouped into three major categories (Pauley, 1993): geometry variables (i.e., tributary drainage area, embankment slope, pile height, etc.), geologic variables (i.e., soil composition, vegetative cover, soil parameters, etc.) and hydrologic variables (i.e., precipitation, evaporation, temperature, etc.). In an effort to build a long-term time averaged model that is capable of predicting gully formation, it is essential to examine the system on a design life time scale and not an event based snapshot in time. Table 3.1 presents a summary of a minimum set of quantitative descriptors necessary for a time averaged approach to gully prediction.

Table 3.1. Summary of Variables

Geometric Variables	Hydrologic Variables	Geologic Variables
• Maximum gully depth • Gully location • Embankment height • Embankment slope • Tributary drainage area • Gully slope • Gully length	• Runoff coefficient • Precipitation depth • Volume of runoff	• Gully age • Soil type (% sands, silts and clays) • Plasticity Index • Vegetative cover • Soil compaction

As observed from Table 3.1, individual precipitation events are not an integral component for a time averaged approach. Justification of this assumption is found in the works of Ireland *et al.* (1939), Bradford *et al.* (1973), Heede (1974, 1976), and Imeson and Kwaad (1980).

In an effort to develop a design procedure to quantify the effects of gullying on an embankment surface, three sources of data were utilized. Both laboratory and field data sets were combined to produce a database with both model and prototype components. The following sections describe the data sets utilized in this analysis.

3.1 LABORATORY SIMULATION

A near-prototype laboratory study was conducted at the Hydraulics Laboratory at Colorado State University to evaluate the gully erosion process into a soil-covered impoundment under controlled conditions. Detailed descriptions of the physical model and testing procedures are presented by Pauley (1993). A summary of the experimental investigation is presented in the following sections.

3.1.1 Physical Model

In order to correlate the effects of soil type, embankment geometry and hydrologic activity with gully development, an outdoor flume was modified to simulate the rainfall/runoff processes on a constructed soil embankment (Pauley, 1993). The flume was 20-ft wide, 8-ft deep, and 100-ft long. A schematic of the simulation facility is presented in Figure 3.1. The flume was equipped with a moveable carriage and point gage measurement system with operational accuracy of 0.2 inches in the vertical and 0.5 inches in the longitudinal and lateral directions.

A sprinkler system capable of simulating rainfalls up to 10 in./hr was installed around the perimeter of the flume. Flow was measured using a calibrated orifice plate with a differential manometer. A headbox was installed adjacent to the flume head wall to simulate tributary overland flows and to simulate flow on the embankment. A riprap transition was constructed between the headbox and the embankment. The sprinkler and headbox system provided a simulation capacity equivalent to 60 in./hr.

Seven earthen embankments were constructed and tested in the flume. Soil was placed in approximately 8 in. lifts and compacted with a vibratory roller. Shaping and contouring was performed manually. Embankment geometries were designed to represent typical geometric configurations observed on reclaimed mine and tailings sites located in the western United States. Embankments were constructed 8.5-ft high, top slopes of 2%, and embankment slopes ranging from 11.6 to 20.2%. A schematic plan and profile of the simulated embankments is presented in Figure 3.2. A summary of the model embankment characteristics is presented in Table 3.2.

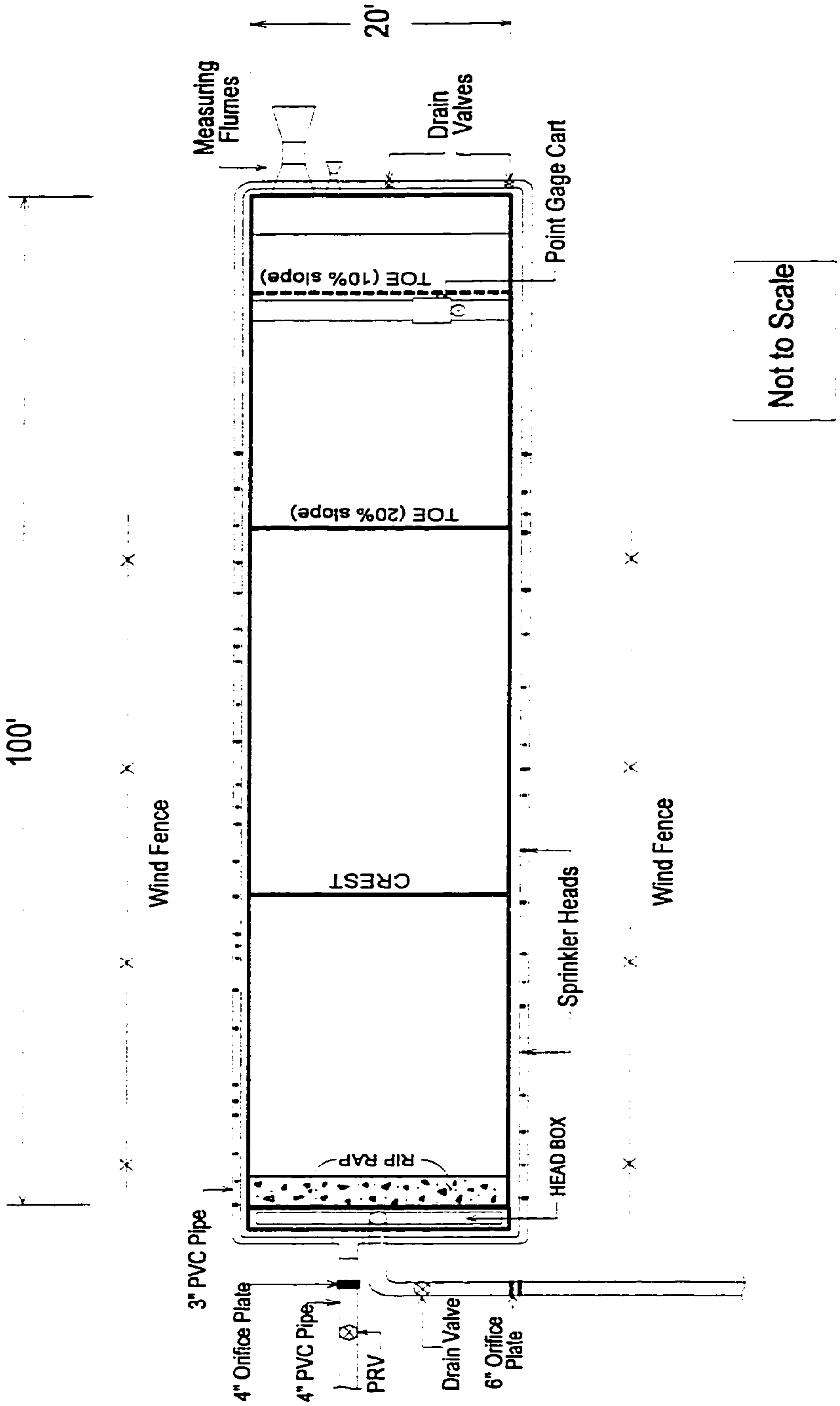

Figure 3.1 Simulation Facility Schematic

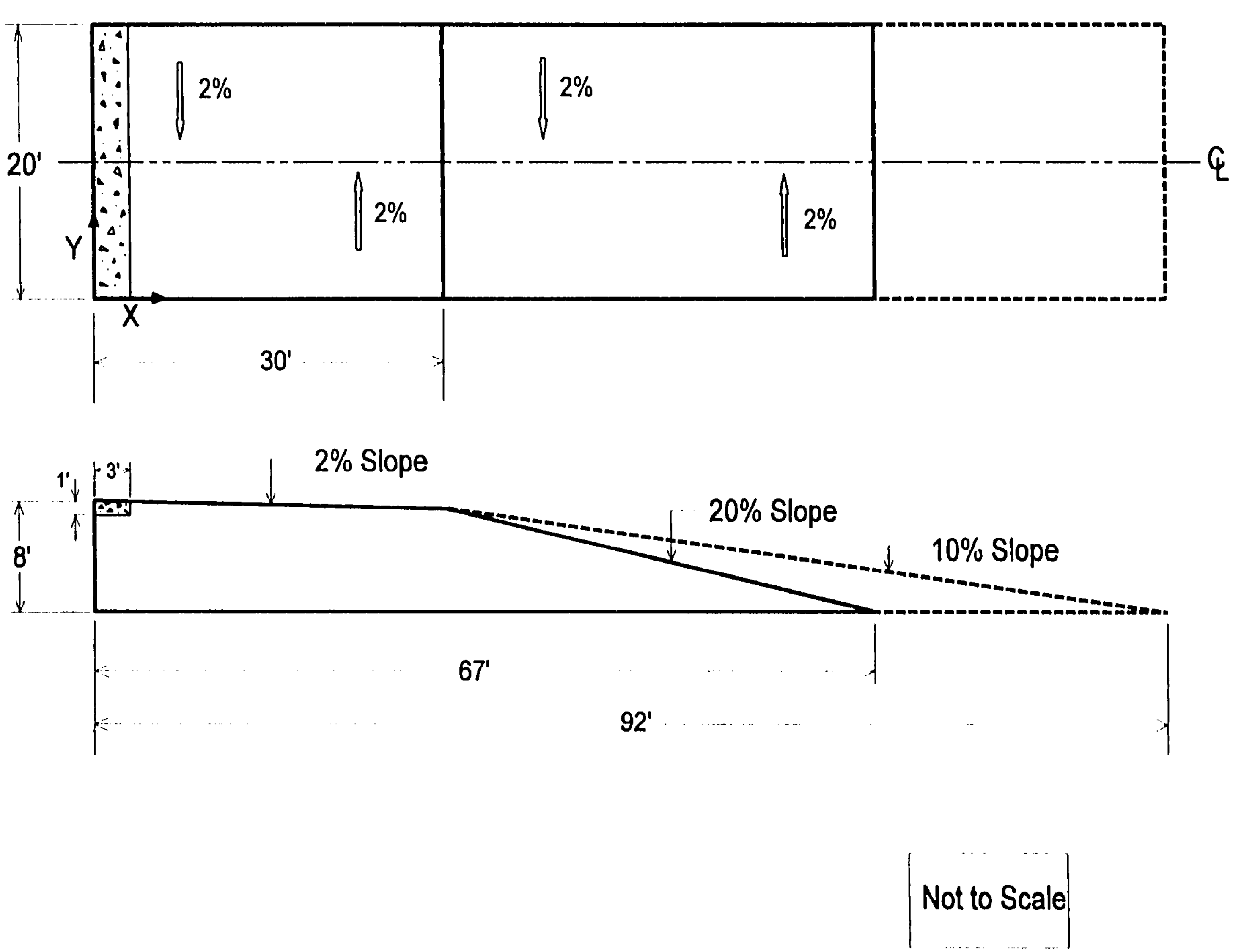

Figure 3.2 Schematic Plan and Profile of the Simulated Embankments

Table 3.2 Summary of Laboratory Embankment Characteristics

Embankment Number	Embankment Height (ft)	Embankment Slope (%)	Compaction (Standard Proctor) (%)
O1	8.0	20.2	86.2
O2	8.0	19.7	78.5
O3	7.7	19.5	80.3
O4	7.6	18.8	89.9
O5	7.6	11.6	79.8
O6	7.7	11.1	91.6
O7	7.6	19.7	95.1

The laboratory embankment soils comprised mixtures of noncohesive sands, and gravels with cohesive silts and clays. The silt/clay to sand/gravel ratios of the embankments were approximately 30:70, 20:80, and 50:50. Table 3.3 summarizes the soil characteristics for each of the seven embankments. Each embankment was uniformly compacted, per Standard Proctor, with compactions ranging from approximately 78.5 to 95.1%.

Table 3.3 Summary of Laboratory Embankment Soil Characteristics

Test No.	Soil Type	% Sand & Gravels	% Silt	% Clay	D_{85} (mm)	D_{50} (mm)	Liquid Limit	Plastic Limit
O1	CL	34.0	17.0	49.0	0.6	0.002	31.2	22.4
O2	CL	28.5	20.0	51.5	0.3	0.002	27.5	19.5
O3	SM	70.0	20.0	10.0	2.0	0.400	0.0	NP
O4	SM	77.5	17.5	5.0	2.2	0.550	0.0	NP
O5	SM	77.5	20.0	2.5	2.4	0.400	0.0	NP
O6	SM	85.0	12.5	2.5	1.8	0.630	0.0	NP
O7	SM	74.3	6.0	19.7	2.0	0.320	0.0	NP

Once each embankment was constructed, a series of hydrological rainfall events was applied to the embankment. The hyetograph was based upon a range of rainfall intensities and total rainfall depths that might be expected in a 200-year period in the arid/semiarid southwest United States. Rainfall intensities ranged from the two-year to the probable maximum precipitation event. Details of the applied hyetograph are presented in Pauley (1993).

3.1.2 Testing Procedure

The testing procedure was consistent for each of the seven embankments. After construction, the embankment was surveyed to establish the pre-rainfall condition. The sprinkler system was then prepared and rainfall was simulated. Contour measurements were recorded for each embankment throughout the application of the prescribed hyetograph. In an effort to replicate natural conditions, the embankment was permitted to drain for a minimum of 12 hours between the nine separate rainfall events applied to each embankment. Upon completion of the prescribed hyetograph and gully measurements, the embankment was removed and the sequential embankment constructed. Data were input into a comprehensive database for subsequent analysis. A summary of the test results is presented in Table 3.4.

3.2 FIELD SITES

Two sets of field data were included in the analysis. The first was obtained by Colorado State University and reported by Abt *et al.* (1995). Falk (1985) investigated

Table 3.4 Summary of Laboratory Embankment Results

Test No.	Sand:Fines Ratio	Embankment Slope (%)	Compaction (%)	Total Runoff (ft^3)	Final D_{max} (ft)	Location D_{max}[1] (ft)
O1	1:4	20	86	103,386	0.58	+28.0
O2	1:4	20	79	108304	3.59	+9.2
O3	4:1	20	80	92460	4.12	0.0
O4	4:1	20	90	117027	3.53	+10.0
O5	4:1	10	80	117027	3.46	+18.0
O6	4:1	10	90	117257	4.47	+14.0
O7	7:3	20	95	234073	3.09	+14.7

[1] Indicates location downslope of embankment crest

methods to predict maximum depths of gully intrusion on reclaimed mine tailings slopes, and a data set of 16 gullies was incorporated into the analysis.

3.2.1 Abt *et al.* (1995)

A field investigation was performed by Abt *et al.* (1995) to locate and measure the physical characteristics of gullies that developed on actual reclaimed mine sites. Based on an extensive search, eleven gully sites were located in the western United States. Each site exhibited at least one gully that was considered well-developed and continuous. Of the 11 embankments included in the field investigation, 6 were located in Wyoming and 5 in Colorado. Figures 3.3 and 3.4 present a general location map for the Wyoming and Colorado sites, respectively. Each site exhibited at least one gully that was considered well-developed and continuous. Field access was limited since current reclamation regulations require owners to perform maintenance on reclaimed sites once gully intrusion reaches a depth of approximately 6 to 12 inches at any location along the embankment

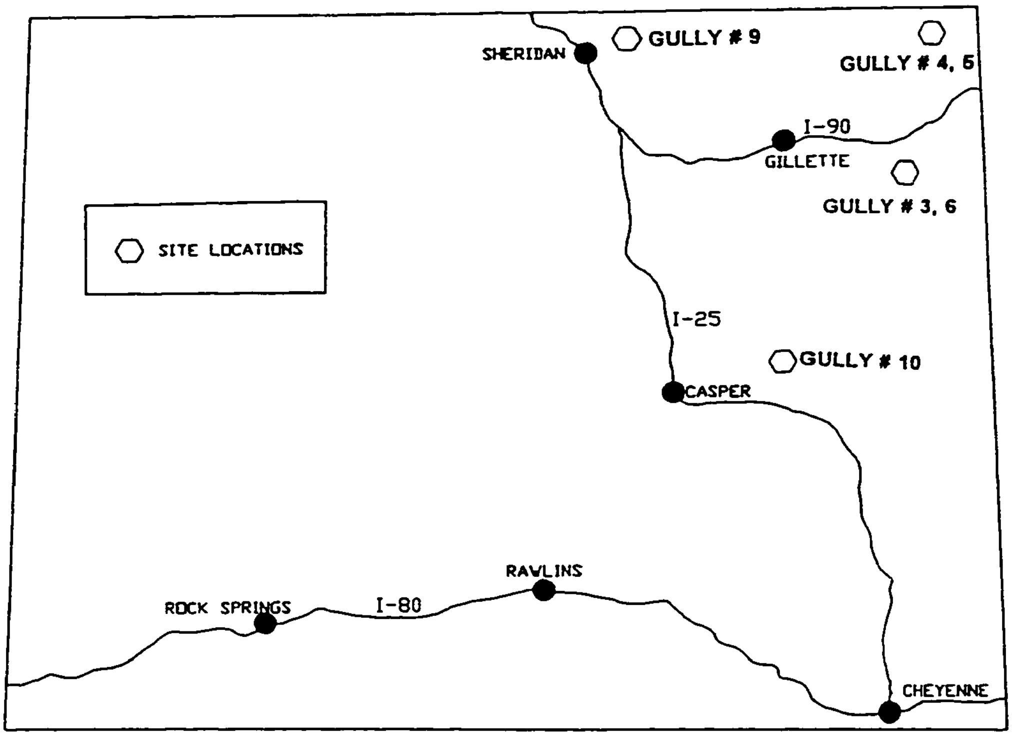

Figure 3.3 Wyoming Field Site Map

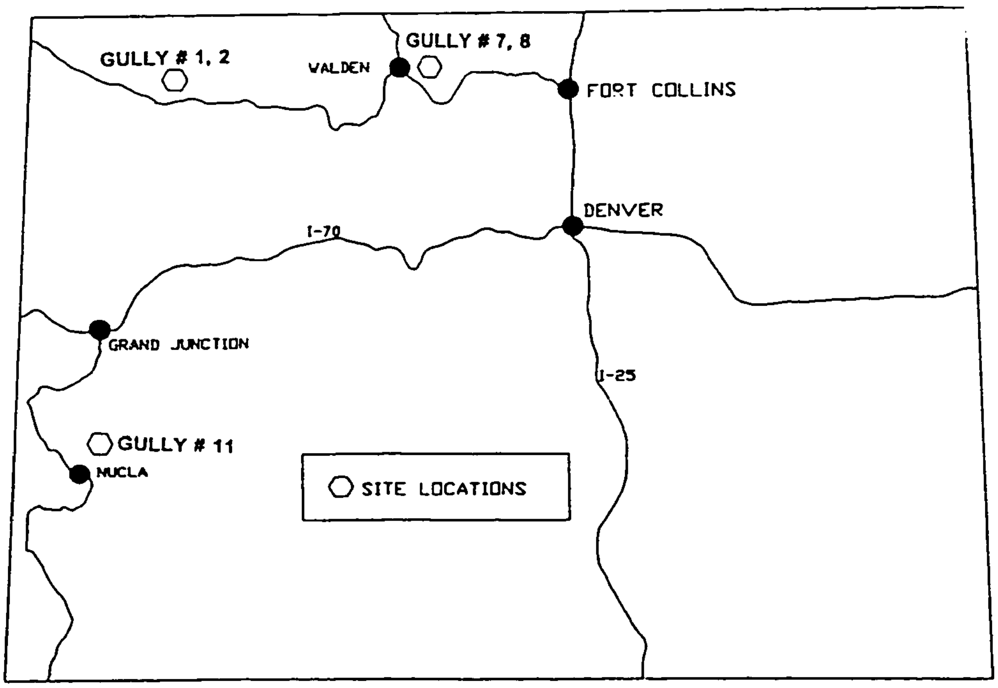

Figure 3.4 Colorado Field Site Map

slope. Owners that were able and willing to cooperate with the field investigation and grant site access, insisted on remaining anonymous and therefore each field site is generically referenced.

Once permission was attained to visit each of the gully sites, data were collected to characterize the embankment slope, the gully, the tributary drainage to the gully, and the cover conditions. An effort was made to collect data in a manner consistent with the laboratory experiments. Quantified variables collected in the field were: the embankment height, initial embankment slope, gully slope, maximum gully depth; and the corresponding width, gully profile, location of the gully maximum depth relative to the embankment crest, soil compaction, and a vegetative cover estimate. A summary of the field measurements is presented in Table 3.5.

Representative soil samples were obtained at each of the 11 sites. The samples were bagged, labeled, and transported to the laboratory for analysis. Soil tests performed included gradation analysis, Atterberg limits analysis, and standard Proctor determination for compaction. The results of the soil analyses are presented in Table 3.6.

Gully development was determined to be dependent primarily upon the runoff volume. The hydrologic parameters needed to determine site runoff are the average annual precipitation and the tributary drainage area. From these parameters, the total volume of runoff tributary to each gully could be estimated based upon the age of the reclaimed impoundment. The average annual precipitation was determined from the U.S. Geological Survey (USGS) rain gages located nearest to the gully site. The runoff was estimated applying the procedures outlined in the SCS (1986) Technical Release 55. Details of the

Table 3.5 Summary of Field Measurements

Site ID	Gully Age (years)	Drainage Area (ft^2)	Initial Slope	Embankment Height (ft)	Gully Length (ft)	Maximum Gully Depth (ft)	Width at D_{max} (ft)	Location of D_{max} Relative to Toe	Vegetative Cover (%)
G-1	12	33,541	0.16	29	203	7.20	15.00	0.57	25
G-2	30	40,511	0.38	74	336	13.50	16.50	0.27	15
G-3	12	1,965,000	0.03	5	292	2.40	2.20	0.76	60
G-4	5.5	15,753	0.24	27	165	7.30	18.00	0.62	5
G-5	4.5	15,983	0.10	6	101	2.70	9.50	0.41	10
G-6	4.5	3,944	0.13	10	60	1.20	3.00	0.75	40
G-7	8	191,740	0.11	20	216	1.50	3.00	0.43	50
G-8	8	51,166	0.22	25	190	2.70	3.50	0.35	35
G-9	5	649,994	0.46	23	110	6.50	5.00	0.50	20
G-10	14	46,415	0.45	175	430	8.70	10.50	0.79	15
G-11	19	5,156	0.70	53	188	2.80	6.00	0.37	5

Table 3.6 Summary of Field Site Soil Characteristics

Site No.	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	D_{50} (mm)	Compaction (Standard Proctor) (%)
G-1	21.5	0	21.5	0.3	87
G-2	18.8	0	18.8	0.3	65
G-3	37.8	26.6	11.2	1.1	77
G-4	36.8	28.7	8.2	4.0	70
G-5	46.6	33.9	12.7	2.7	71
G-6	40.0	28.2	11.8	2.4	72
G-7	37.6	20.8	16.8	1.2	75
G-8	29.6	20.2	9.4	1.0	79
G-9	43.1	24.1	19.0	1.1	81
G-10	39.0	22.6	16.4	0.7	50
G-11	27.5	20.2	7.3	1.3	49

hydrologic analysis are presented by Abt *et al.* (1995). A summary of the hydrologic parameters and total estimated volume of runoff is presented in Table 3.7.

3.2.1.1 Site Descriptions

Each of the 11 sites reported by Abt *et al.* (1995) contained gullies formed into reclaimed surfaces. A brief description of each site is provided in Appendix A to document conditions typically found on reclaimed mine sites and to demonstrate the necessity of a methodology aimed at predicting the erosion potential of reclaimed sites over time.

3.2.2 Falk (1985)

Falk (1985) reported on 16 gullies on reclaimed mine sites in south central Wyoming and north central Colorado. Primary considerations in the selection of sites was

Table 3.7 Summary of Field Site Hydrological Characteristics

Gully ID	Site Location	Weather Recording Station	Gully Age (years)	Hydrologic Soil Group	SCS Curve Number	Total Runoff (ft^3)
G-1	Maybell, Colorado	Maybell, Colorado	12	B	79	7,038
G-2	Maybell, Colorado	Maybell, Colorado	30	B	79	16,184
G-3	Maybell, Colorado	Upton, Utah	12	C	89	3,723,020
G-4	Maybell, Colorado	Bell Fourche, S. Dakota	5.5	C	86	6,327
G-5	Maybell, Colorado	Bell Fourche, S. Dakota	4.5	D	86	6,153
G-6	Maybell, Colorado	Upton, Wyoming	4.5	C	89	1,729
G-7	Maybell, Colorado	Walden, Colorado	8	C	86	29,464
G-8	Maybell, Colorado	Walden, Colorado	8	C	86	7,863
G-9	Maybell, Colorado	Sheridan, Wyoming	5	B	79	30,821
G-10	Maybell, Colorado	Glenrock, Wyoming	14	C	86	40,277
G-11	Maybell, Colorado	Uravan, Colorado	19	C	86	1,160

the presence of gully formation on the reclaimed or abandoned surface of milling and mining operations and the willingness of the owners to grant access.

Data collected at each site included: measurements of maximum gully depth, the width at the location of maximum depth, total gully length, embankment slope, drainage area, gully bottom slope, gully age, and an estimate of the vegetative cover. In addition, soil samples were collected at each site and transported back to the laboratory for analysis. Atterberg Limit tests and grain size analysis were performed on each soil sample. Samples were not collected at sites D, G and D-7. Soil samples collected from the study sites were classified as poorly graded to well graded sand and silty sand. Due to the low value of the Plasticity Index and the average particle size (D_{50}), the soils were considered non-cohesive in nature.

Since climatological data were not available for each site, the area containing all 16 study sites was divided into 5 areas. Rainfall data collected from four gage locations in Wyoming and one in Colorado were then used to regionalize the precipitation data for the study sites. Figure 3.5 presents a schematic of the climatological regions reported by Falk (1985). Data collected at each of the 5 gages were examined and the number of storms producing 0.5 inches of rainfall or greater were tabulated over the life of the gully. Runoff was calculated for each event and the results summed to produce to total volume of runoff to pass through each of the 16 gullies. Table 3.8 presents a summary of the soil and climatological analysis. Table 3.9 presents a summary of the physical description of each site examined by Falk (1985).

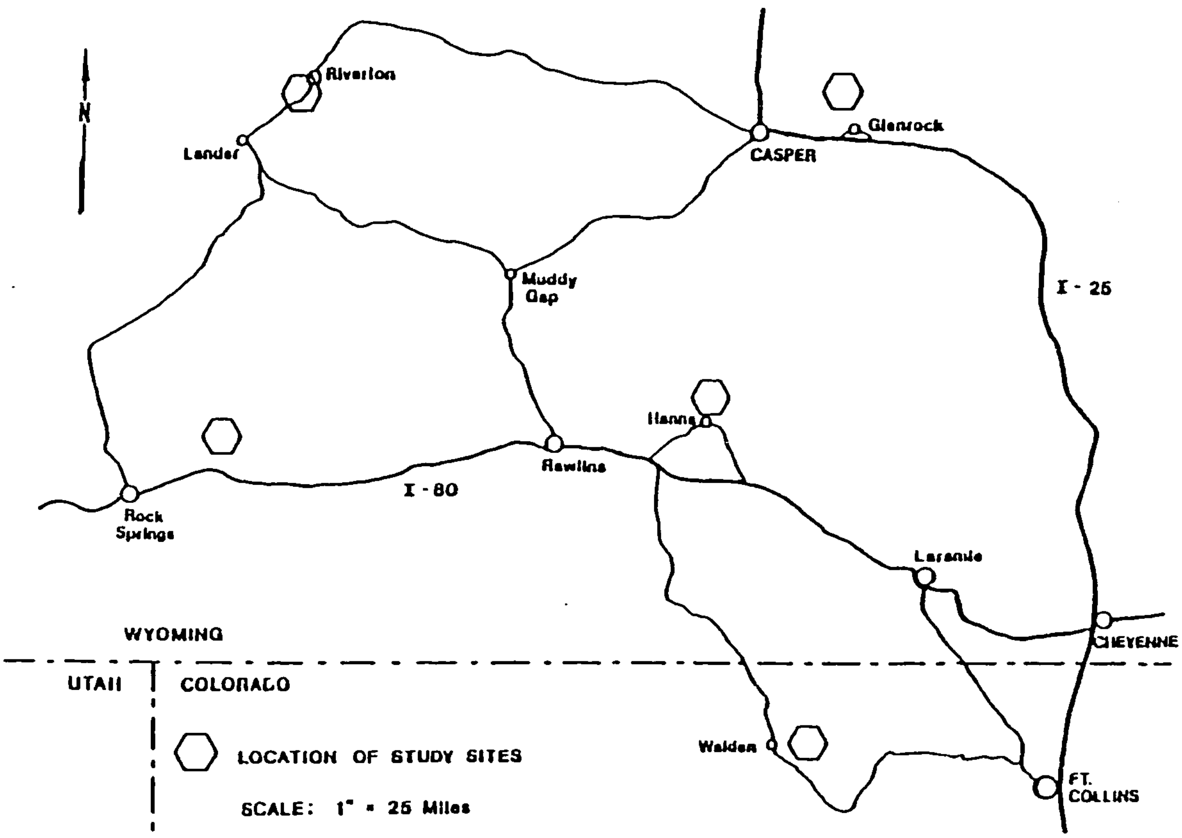

Figure 3.5 Schematic of the Climatological Regions Reported by Falk (1985)

Table 3.8 Summary of Soil and Runoff Data

Site	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	D_{50} (mm)	Average Annual Runoff (in)	Total Runoff (ft^3)
A	22.4	21.3	1.1	0.36	0.2	98
B	28.4	19.8	8.6	0.65	0.2	150
C	31.8	19.7	12.1	1.00	0.2	7.010
D	---	---	---	---	0.6	216
E	21.4	19.9	1.5	0.36	0.6	906
F	30.8	23.1	7.7	0.55	0.6	1,411
G	---	---	---	---	0.6	467
H	29.9	23.8	6.1	2.00	0.6	3,364
I	27.8	24.0	3.8	1.74	0.6	510
J	22.6	21.4	1.2	0.79	0.6	1,042
K	34.2	18.7	15.5	1.82	10	23,945
L	20.2	---	0.0	0.36	1	839
M	22.5	21.1	1.4	0.54	0.5	1,289
N	17.9	---	0.0	0.46	0.5	130,000
O	21.0	19.1	1.9	1.60	0.5	212
D-7	---	---	---	---	0.6	11,577

Table 3.9 Summary of Field Sites Reported by Falk (1985)

Site ID	Gully Age (years)	Drainage Area (ft^2)	Initial Slope	Embankment Height (ft)	Gully Length (ft)	Maximum Gully Depth (ft)	Width at D_{max} (ft)	Location of D_{max} relative to toe (%)	Vegetative Cover (%)
A	7	840	0.23	49	210	2.87	3.12	0.301	35
B	11	819	0.21	39	181	1.25	2.4	0.249	30
C	12	35,048	0.17	30	174	3.5	3.97	0.397	5
D	15	288	0.67	100	144	1.82	2.2	0.5487	5
E	34	533	0.60	42	67	1.7	1.6	0.562	30
F	7	4,032	0.29	60	203	1.2	2.6	0.525	40
G	7	1,335	0.14	24	166	0.6	5.2	0.316	20
H	13	5,176	0.40	97	248	3.17	4.27	0.163	30
I	10	1,021	0.15	29	191	0.6	1.86	0.361	10
J	10	2,085	0.66	155	222	2.7	3.7	0.265	5
K	6	4,789	0.69	83	117	3.87	4.83	0.794	10
L	19	530	0.29	15	49	2.25	5.77	0.768	5
M	8	3,869	0.62	129	199	1.07	1.47	0.277	35
N	8	390,000	0.17	135	815	2.9	3.4	0.556	15
O	8	637	0.29	38	128	0.4	0.85	0.251	15
D-7	15	15,437	0.43	88	203	4.1	4.7	0.317	10

CHAPTER 4

ANALYSIS AND RESULTS

4.1 GENERAL ANALYSIS

Empirical analyses were conducted to consolidate the data derived from the seven outdoor gully simulations from Pauley (1993), the 16 data sets from Falk (1985), and from the eleven field sites reported by Abt *et al.* (1995). Data, corresponding to the variables outlined in Table 3.1, from each of the 34 data sets were identified and assembled into a database. Table 4.1 presents a summary of the data used in the analysis.

An additional analysis of the equations presented by Abt *et al.* (1995) has resulted in a design evaluation procedure for embankment types most commonly encountered in the remediation of low level hazardous waste sites. These analyses address the predication of the maximum depth of incision, the width at the location of maximum depth, and the location of the point of maximum depth with respect to the crest of the slope. In addition, a conservative approach to evaluate the long-term (> 1,000 years) stability of the waste cell is presented.

A fundamental dimensional analysis was performed on the variables presented in Table 4.1 in an effort to develop dimensionless relationships capable of predicting the depth, location, and width of a gully forming on an embankment. Variables were

Table 4.1 Summary of Database

Gully ID	V_f (ft^3)	D_{max} (ft) (ft)	Width (ft)	D_t (ft)	S_o (%)	S_b (%)	H_o (ft)	L_o (ft)	X_o (ft)	Clay (%)
O1	103,386	0.58	1.80	28.00	20.2	7.2	8	40	39.60	49
O2	108,304	3.59	7.06	9.20	19.7	8.4	8	41	40.61	51.5
O3	92,460	4.12	9.46	0.00	19.5	5.6	7.7	40	39.49	10
O4	117,027	3.53	2.64	10.00	18.8	4.1	7.6	41	40.43	5
O5	117,027	3.46	8.69	18.00	11.6	3.6	7.6	66	65.52	2.5
O6	117,257	4.47	4.43	14.00	11.1	9.1	7.7	70	69.37	2.5
O7	234,073	3.09	N/A	14.70	19.7	5.1	7.6	40	38.58	19.7
G-1	7,038	7.20	15.00	87.29	16.0	9.7	29	203	181.25	12.1
G-2	16,184	13.50	16.50	245.28	38.0	22.0	74	336	194.74	16
G-3	3,723,020	2.40	2.20	70.08	3.0	2.1	5	292	166.67	58
G-4	6,327	7.30	18.00	62.70	24.0	16.7	27	165	112.50	58
G-5	6,153	2.70	9.50	59.59	10.0	3.4	6	101	60.00	61
G-6	1,729	1.20	3.00	15.00	13.0	10.3	10	60	76.92	76
G-7	29,464	1.50	3.00	123.12	11.0	9.4	20	216	181.82	57
G-8	7,863	2.70	3.50	123.50	22.0	17.8	25	190	113.64	42
G-9	30,821	6.50	5.00	55.00	46.0	33.0	23	110	50.00	43
G-10	40,277	8.70	10.50	90.30	45.0	42.2	175	430	388.89	40
G-11	1,160	2.80	6.00	118.44	70.0	65.1	53	188	75.71	33
A	98	2.87	3.12	146.79	23.0	18.3	49	210	213.04	5.9
B	150	1.25	2.40	135.93	21.0	18.4	39	181	185.71	2.3
C	7,010	3.50	3.97	104.92	17.0	11.6	30	174	176.47	2.7
D	216	1.82	2.20	64.99	67.0	64.1	100	144	149.25	N/A
E	906	1.70	1.60	29.35	60.0	54.9	42	67	70.00	2.5
F	1,411	1.20	2.60	96.43	29.0	27.5	60	203	206.90	0.6
G	467	0.60	5.20	113.54	14.0	12.5	24	166	171.43	N/A
H	3,364	3.17	4.27	207.58	40.0	31.5	97	248	242.50	3.3
I	510	0.60	1.86	122.05	15.0	14.3	29	191	193.33	0.3
J	1,042	2.70	3.70	163.17	66.0	60.8	155	222	234.85	2.4
K	23,945	3.87	4.83	24.10	69.0	63.6	83	117	120.29	6.7
L	839	2.25	5.77	11.37	29.0	22.3	15	49	51.72	0.3
M	1,289	1.07	1.47	143.88	62.0	59.9	129	199	208.06	0.5
N	130,000	2.90	3.40	361.86	17.0	16.0	135	815	794.12	0.4
O	212	0.40	0.85	95.87	29.0	28.0	38	128	131.03	0.2
D-7	11,577	4.10	4.70	138.65	43.0	36.0	88	203	204.65	N/A

systematically combined to obtain a comprehensive set of dimensionless parameters. Equation (4.1) presents a summary of the parameters examined in the analysis.

$$F\left[\frac{D_{max}}{H_o}, \frac{D_{max}}{X_o}, \frac{V_r}{H_o^3}, \frac{V_r}{H_oL_o^2}, \frac{V_r}{H_o^2L_o}, \frac{D_{max}}{L_oS_o}, \frac{D_{max}}{X_oS_o}, \frac{V_r}{L_o^3}\right] = 0 \qquad (4.1)$$

where: D_{max} = the maximum depth of gully incision (m);

H_o = the embankment pile height (m);

S_o = the original embankment slope (%);

V_r = the cumulative volume of runoff (m^3);

L_o = the slope length (m); and

X_o = the horizontal embankment length (m).

Each parameter and combinations of parameters, were plotted and analyzed to determine a set of design equations capable of predicting the potential depth, location, and width of a gully forming on an earthen embankment.

4.2 MAXIMUM GULLY INCISION

The maximum depth of gully incision (D_{max}) was estimated as a function of the cumulative volume of runoff (V_r), the embankment height (H_o), embankment slope length (L_o), the embankment slope (S), and the clay content of the construction material. Through dimensional analysis, the ratio of the maximum depth of incision to the product of the slope angle (percent), and slope length was correlated to the ratio of the cumulative volume divided by the cube of the embankment (pile) height as presented in Figure 4.1. It was observed that the data were widely scattered such that a single best-fit relationship was not practical. A series of nonlinear envelope curves was fit to the data as shown in

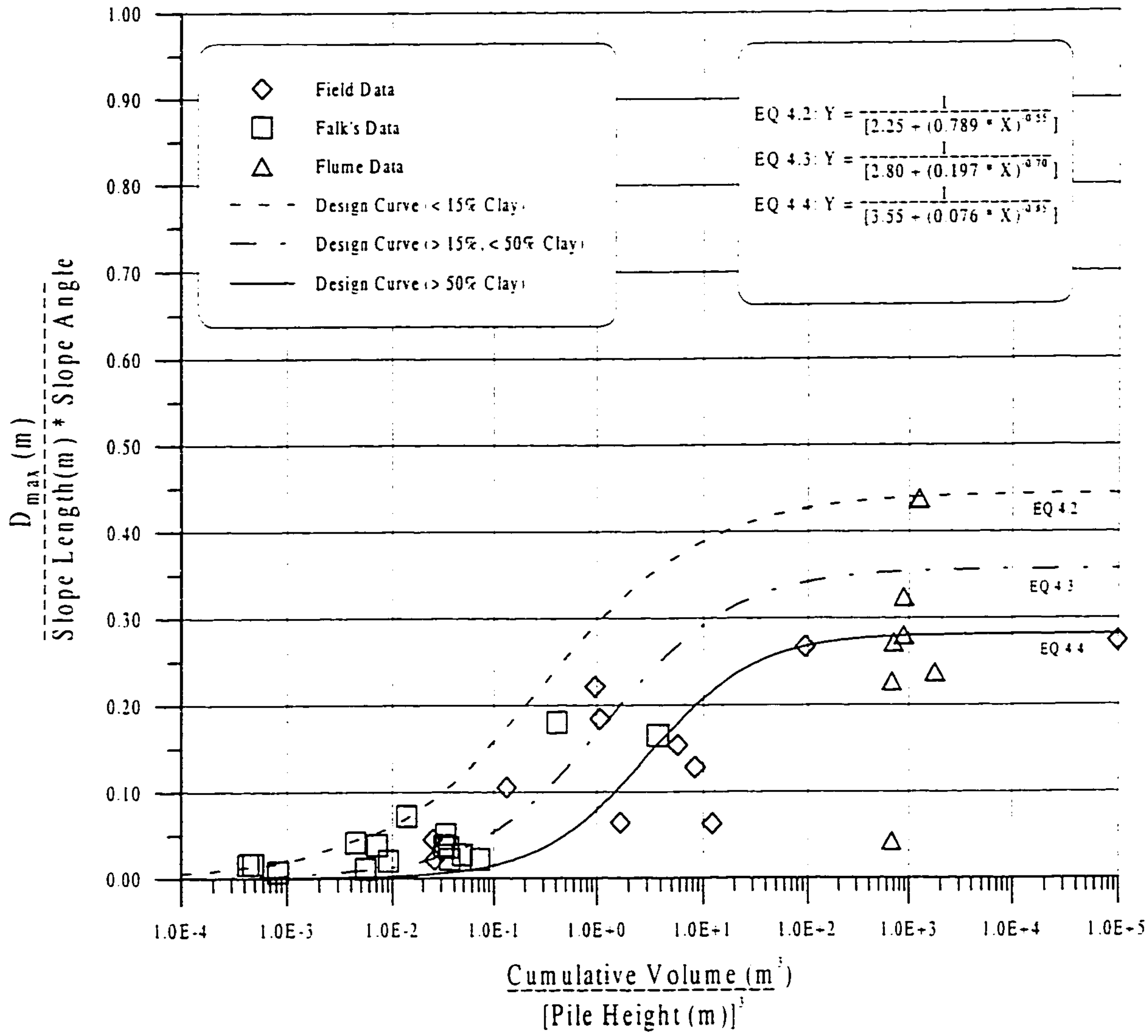

Figure 4.1 Envelope Curves Developed to Estimate D_{max}

Figure 4.1. The envelope curves were derived from Imeson and Kwaad (1980) who developed a nonlinear multiple regression exponential model for relating primary variables into a S-shaped relation, which is applicable to the erosive process. Since any breach of the embankment/cover may result in exposure of the waste (not considered acceptable), an envelope curve fit approach was taken to provide a conservative estimate of the dependent variable.

One of the variables the designer often cannot control in the construction process is the soil composition. Therefore, the envelope relationships presented in Figure 4.1 correspond to the site specific soil clay content. The upper curve reflects soil compositions that contain 15% or less of clay. The premise is that the lower the clay content, the greater the potential susceptibility to the gully erosive processes (Abt *et al.*, 1995). The middle curve corresponds to a clay content greater than 15% but less than 50% while the lower curve indicates that the clay content is greater than 50%. Embankments comprised of a high percentage of clay (greater than 50%) demonstrate a high resistance to erosion.

Three dimensionless relationships were derived to estimate the maximum depth of gully incision on a reclaimed tailings embankment/cover as a function of the cumulative volume of runoff, embankment/pile height, slope length, and slope angle. The resulting expressions for estimating the maximum depth of gully incision are:

Clay content less than 15%:

$$\frac{D_{max}}{L_o * S_o} = \frac{1}{2.25 + \left(0.789 * \frac{V_r}{H_o^3}\right)^{-0.55}} \qquad (4.2)$$

Clay content greater than 15% but less than 50%:

$$\frac{D_{max}}{L_o * S_o} = \frac{1}{2.80 + \left(0.197 * \frac{V_r}{H_o^3}\right)^{-0.70}} \tag{4.3}$$

Clay content greater than 50%:

$$\frac{D_{max}}{L_o * S_o} = \frac{1}{3.55 + \left(0.076 * \frac{V_r}{H_o^3}\right)^{-0.85}} \tag{4.4}$$

Each envelope curve portrays the three proposed growth phases of a gully: the initial development, the accelerated growth, and the healing/stabilization as suggest by the SCS (1966). The initial development of the gully is the relatively flat portion at the base of the curve. The accelerated growth phase is exemplified by the steep, rising limb of the curve. The healing/stabilization phase of incision is the flattening curve segment near the top of each relation.

A gully factor, G_f, was developed from the expressions presented in Equations (4.2), (4.3), and (4.4) for varying clay content of the proposed construction material. The gully factor is defined as:

$$G_f = \frac{D_{max}}{L_i * S_o} \tag{4.5}$$

where: G_f = the gully factor;

D_{max} = the maximum depth of gully incision (m);

L_i = the slope length (m); and

S = the embankment slope (%).

4.3 LOCATION OF MAXIMUM INCISION

In many applications, it is important to estimate the location of the maximum gully incision to evaluate the stability of the embankment/cover and the potential for a penetration into the waste material. The location of D_{max} was recorded for 105 data sets. However, 73 of the 105 data sets were derived from mid-test measurements. Therefore, 32 data sets were related to the location of maximum gully incision on the embankment/cover. The location of maximum incision was converted from a length to a scalar expressed in terms of the number of D_{max}'s up slope or downslope from the embankment crest. In 31 of the 32 data sets, the maximum depth of incision was located at or downslope of the embankment crest.

A dimensionless expression was derived by correlating the location of D_{max} (downslope of the crest) to the ratio of the product of the cumulative runoff volume and slope angle (decimal) divided by the cube of the slope length as presented in Figure 4.2. The location of the maximum depth of incision may be estimated by:

$$D_{\ell} = 0.713 * \left(\frac{\left(V_r * S_o\right)}{L_o^3} \right)^{-0.415} \qquad (4.6)$$

where D_{ℓ} is the multiple of D_{max} downslope from the embankment crest.

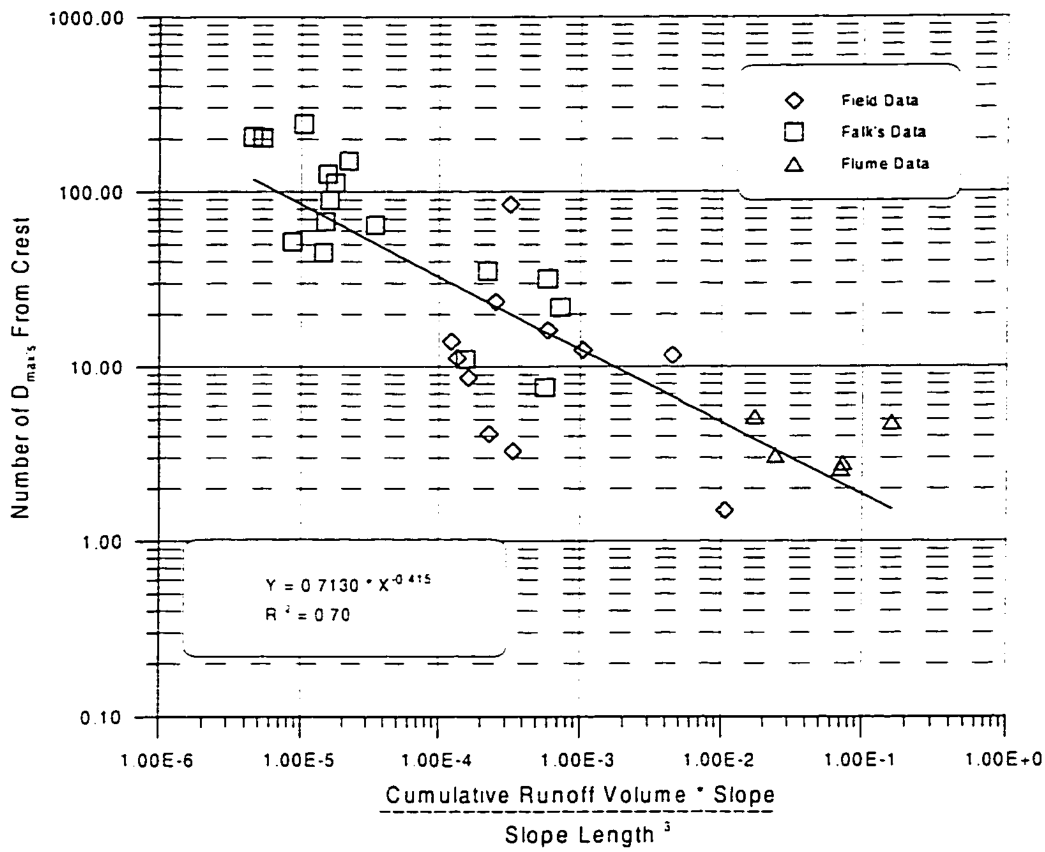

Figure 4.2 Data Used to Generate Equation (4.6)

The laboratory experiments simulated approximately 200 years of erosion with limited weathering while the oldest gully observed in the field was 30 years old. Based upon the observations, expecting that D_{max} occurs at or downslope of the embankment crest for evaluation periods of 200 years or less is reasonable.

4.4 GULLY TOP WIDTH

In many applications, determining the top width of the gully may be useful for estimating the gully widening impact on the embankment stability or for estimating sediment loss through the erosive processes. Gully top widths (W) at the location of D_{max} were obtained from the laboratory experiments, Falk's (1985) field measurements, and the study field sites. The gully top widths were correlated to D_{max} as illustrated in Figure 4.3. A power regression relation was fit to the data for the best-fit scenarios resulting in the predictive expression

$$W = \sqrt[0.87]{\frac{D_{max}}{0.61}} \tag{4.7}$$

where the top width (W) is in meters. An envelope relation was also fit to the data to provide a conservative approximation of the gully width at the point of maximum gully incision. The envelope may be expressed as:

$$W = \sqrt[0.7]{\frac{D_{max}}{1.60}} \tag{4.8}$$

The selection of the desired estimation relationship depends upon the conservatism of the user.

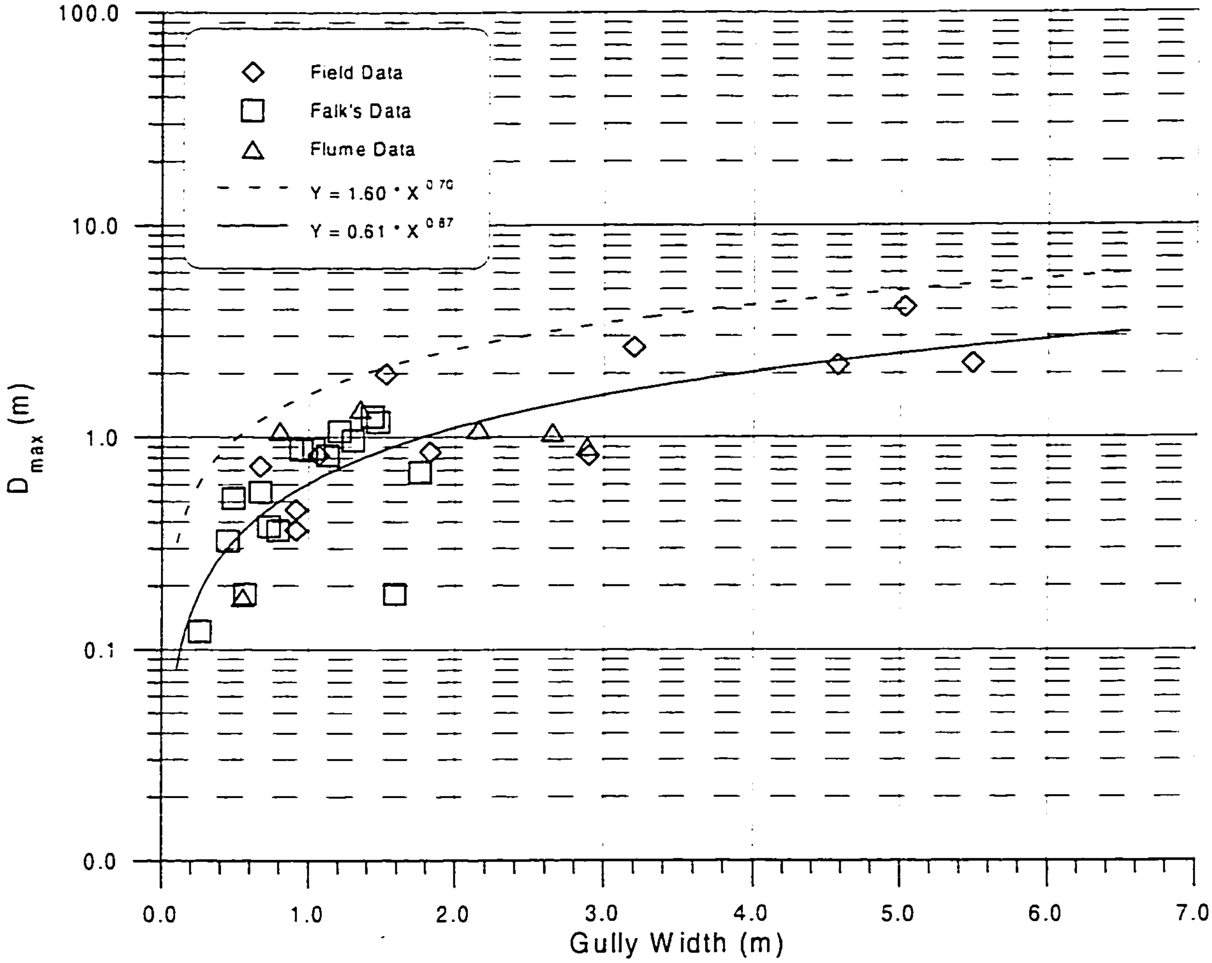

Figure 4.3 D_{max} Versus Gully Width

4.5 SLOPE EVOLUTION

It was observed in each of the flume simulations that as the gully developed on the embankment slope, the gully bed (thalweg) slope (S_b) flattened as the volume of runoff through the gully increased. The observed gully bed slopes for the seven flume tests (end of test results), the field gullies, and field gullies presented by Falk (1985) were plotted against the original embankment gradient (S_o) as presented in Figure 4.4. Knowing the original embankment slope gradient, it is possible to estimate the gully bed slope that may develop as:

$$S_b = [1.008 * S_o] - 0.063 \qquad (4.9)$$

It is observed that gully bed slope flattens approximately 15% from the original embankment slope gradient. The gully bed slopes were derived from gullies incised into embankment slopes that existed for 4.5 to 30 years and model embankments with approximately 200 years of runoff. Although it is reasonable to assume that the gully bed slopes will continue to flatten for periods beyond 200 years, insufficient data are available to assess the continued reduction in slope.

4.6 POTENTIAL TRIBUTARY DRAINAGE AREA

The gully growth and development is assumed to be a function of the total volume of runoff tributary to the gully. Site runoff is estimated by determining the potential tributary drainage area to the toe of the slope. The drainage area tributary to the toe is the sum of the external basins that exist up slope of the embankment and the drainage areas that develop on the embankment/cover. Since the focus of this procedure is to estimate the

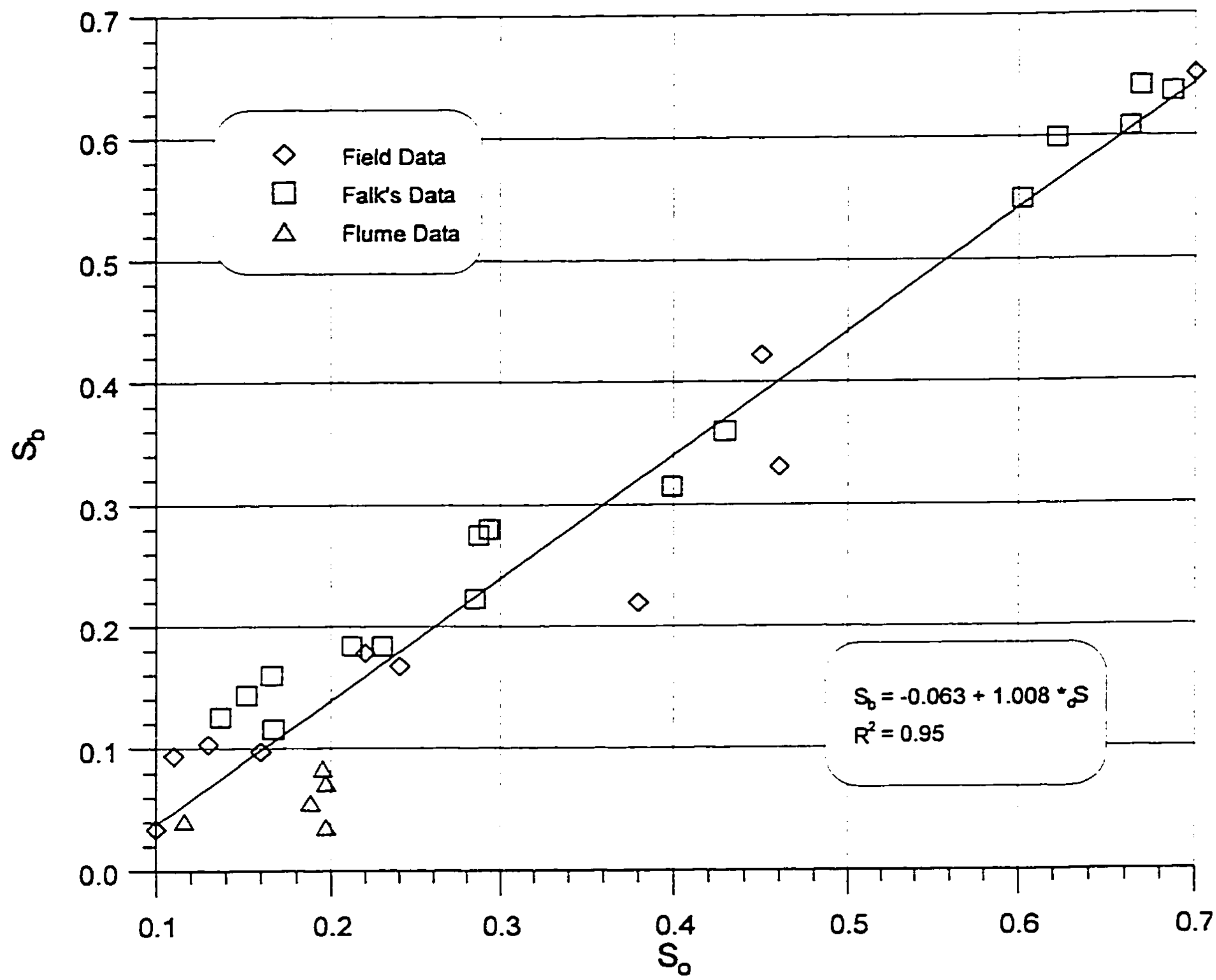

Figure 4.4 Data Used to Generate Equation (4.9)

tributary drainage area from the embankment/cover, runoff estimation procedures for drainage basins external to the impoundment will not be addressed.

The drainage area tributary to any point along the toe of the slope is a function of the type of embankment designed. A procedure for estimating tributary drainage area that develops on the embankment/cover will be presented for each of the three embankment types presented in Section 4.8.

A procedure for estimating the potential tributary drainage area at the toe of a uniformly graded, steep slope was developed by Abt *et al.* (1995). The drainage area is estimated as:

$$A = 0.276 * \left[L_o * \cos(\theta)\right]^{1.636} \tag{4.10}$$

where: A = the tributary drainage area (m^2);

L_o = the slope length (m); and

θ = the embankment slope (degrees).

4.7 RUNOFF VOLUME

The development and growth of the gully are assumed to be a function of the total volume of precipitation that collects in the tributary basin and is conveyed through the drainage network for the embankment design life. Three hydrologic elements are needed to compute the potential volume of runoff: the depth of precipitation, the runoff to rainfall ratio, and the design life.

4.7.1 Design Life

The embankment design life (N) must be determined. The design life may be dictated by regulation, costs, or by design use. The embankment design life is estimated based upon the duration, in years, in which little or no maintenance is performed.

4.7.2 Potential Depth of Precipitation

Once the design life is identified, an estimate of the total depth of precipitation (D_t) anticipated during the design life of the project can be determined. Precipitation estimates may be derived from local weather stations, state climatological databases, or from U.S. Weather Bureau isohyetal maps. In an effort to produce a long-term, time averaged approach to estimating gully depth, the average annual precipitation (P) for each of the field sites was used in the analysis. Recognizing the benefit of engineering judgement, a precipitation adjustment coefficient (C_p) is introduced. For typical design analysis, a C_p value of 1.0 is used. However, the designer is able to adjust referenced precipitation values based on site knowledge and experience. The expected depth of precipitation is then estimated as:

$$D_t = N * P * C_p \tag{4.11}$$

where: D_t = the potential depth of precipitation (m);

N = the design life (years);

P = the average annual precipitation (m); and

C_p = the precipitation adjustment coefficient.

4.7.3 Runoff-Rainfall Ratio

The runoff to rainfall ratio, R_r, is needed to convert the potential depth of precipitation for the embankment design life to potential runoff tributary to the developing gully. The USGS developed a runoff map method (Gebert *et al.*, 1989) to determine the average annual runoff expected from any location in the United States. The USGS map provides the user the annual depth of runoff from a site specific location. The ratio of the runoff to rainfall is computed by dividing the runoff depth derived from Gebert *et al.* (1989) by the average annual precipitation for the appropriate locale. The average runoff ratio using the USGS Average Annual Runoff Method is 0.127. The runoff-rainfall ratio of 0.127 provides an estimate for the arid and semi-arid regions of the western United States.

4.7.4 Cumulative Volume of Runoff

The cumulative volume of runoff (V_r) tributary to the embankment toe, in cubic meters, is calculated as:

$$V_r = D_t * R_r * A \quad (4.12)$$

where: V_r = the cumulative volume of runoff (m^3);

D_t = the potential depth of precipitation;

R_r = the runoff-rainfall ratio; and

A = the tributary drainage area (m^2).

It is acknowledged that a single storm event will significantly impact the development of the gully. However, over a 1,000-year period the volume of runoff tributary to the gully for the embankment design life is the primary element reflecting the analysis period.

4.8 EMBANKMENT CONFIGURATIONS

A review of existing waste and tailings reclamation designs in conjunction with numerous site visits indicates that three primary embankment/cover configurations are commonly proposed. The three embankment configurations or types have been proposed or constructed as presented in Figures 4.5a, 4.5b, and 4.5c.

Embankment Type 1 is characterized by an embankment slope, commonly 1:10 to 1:20 (V:H), that transitions at the crest to a flat top as indicated in Figure 4.5a. The characteristic variables are the original slope length, L_o, the original embankment height, H_o, the embankment slope angle, θ, and the original slope base length, X_o. Type 1 embankments are not commonly proposed or observed but remain a design option.

Type 2 embankments are commonly proposed for the encapsulation of nuclear waste and uranium tailings impoundments. The structure is characterized by the embankment slope transitioning to a relative flat ($S < 5\%$) top slope. The top slope drains toward the main embankment as illustrated in Figure 4.5b. Design variables include those presented in the Type 1 embankment in addition to the top slope length, L_2, the top slope height, H_2, and the top slope angle, S_2. The Type 2 embankment is the most common configuration proposed at this time.

The geometry of the Type 3 embankment is characterized by an adverse top slope transitioning into the main embankment at the crest as presented in Figure 4.5c. The

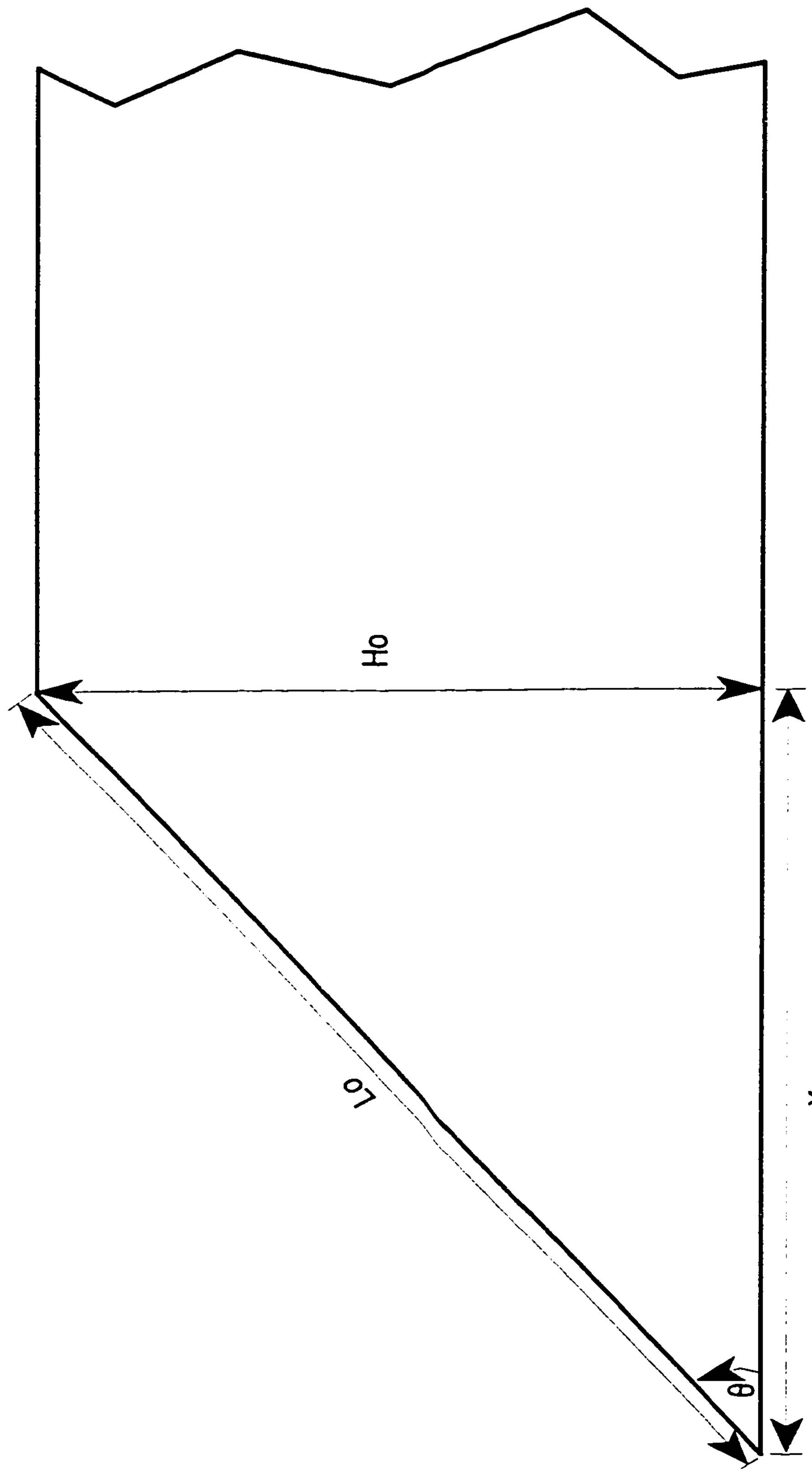

Figure 4.5a Type 1 Embankment

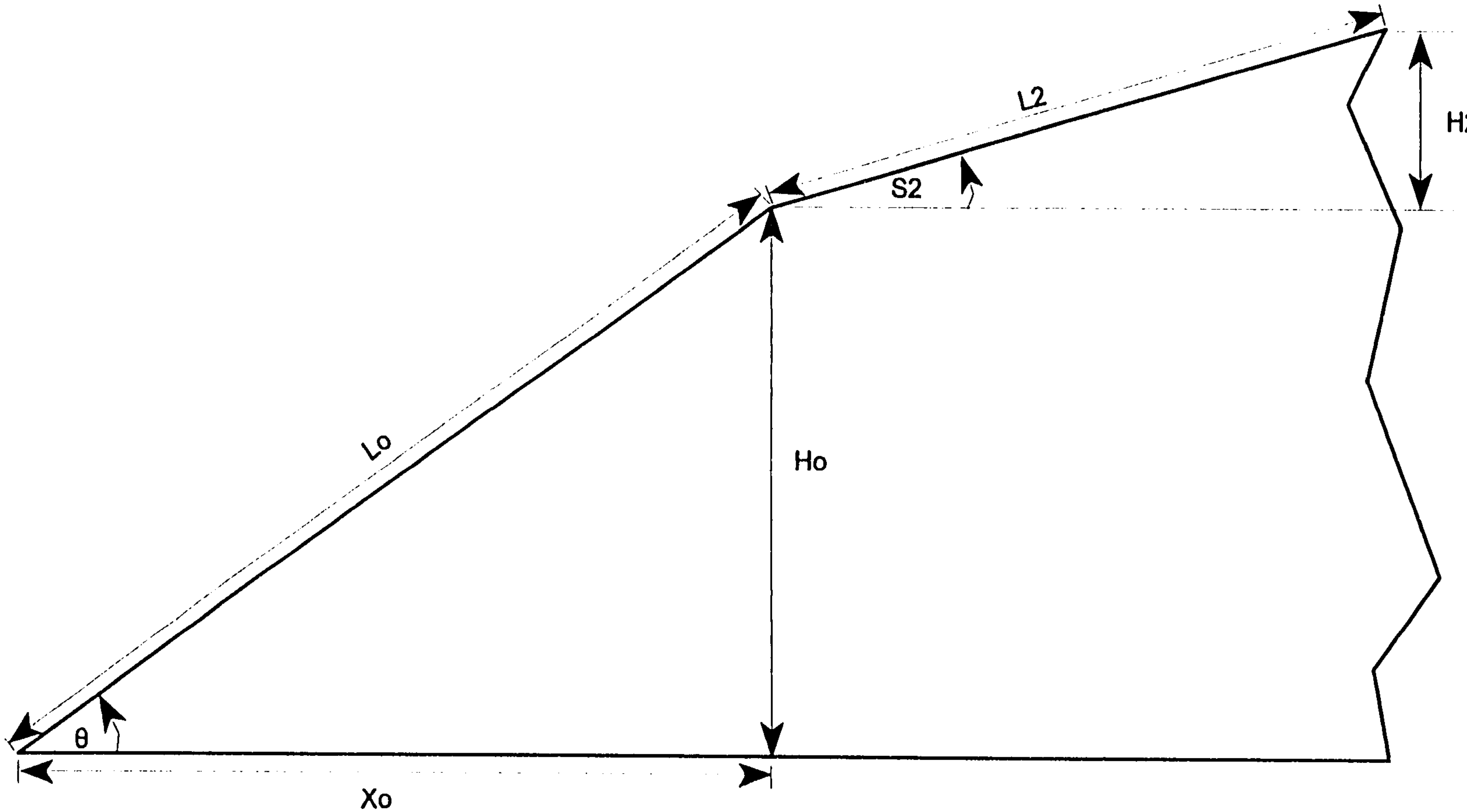

Figure 4.5b Type 2 Embankment

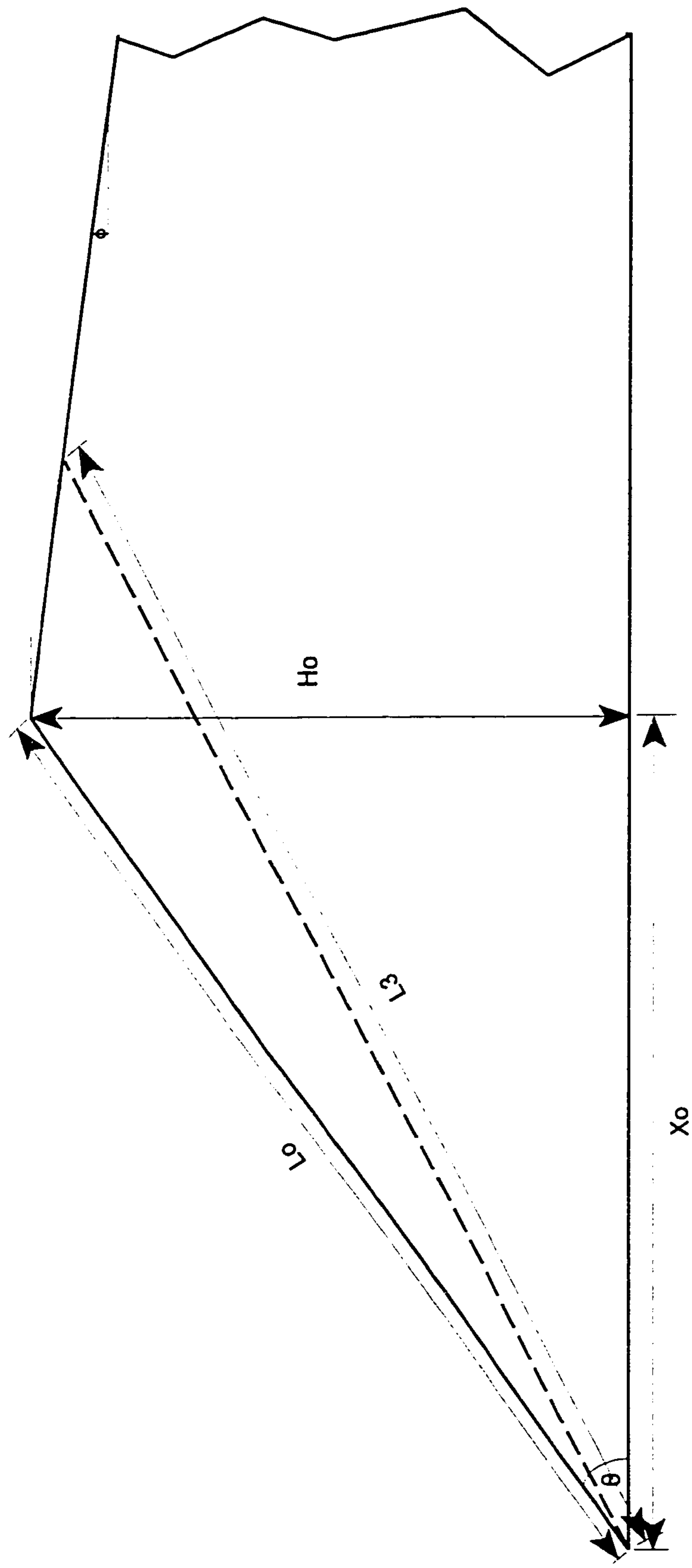

Figure 4.5c Type 3 Embankment

primary design variables include those presented for embankment Type 1 and the transitional embankment slope length, L_3. The Type 3 configuration is not commonly used but is a design option.

It is important to recognize that although each embankment type is similar along the main embankment face, the top slope, and subsequent potential tributary drainage, significantly impacts the maximum depth of gully incision, D_{max}, that may intrude into the embankment face. Therefore, a unique procedure must be developed in order to estimate the potential tributary drainage area and volume of runoff for each embankment type.

4.8.1 Type 1 Embankment

The Type 1 embankment is characterized by a side slope that transitions at the crest to a flat top or cap as presented in Figure 4.5a. The drainage area of a Type 1 embankment is estimated applying Equation (4.10).

4.8.2 Type 2 Embankment

The Type 2 embankment is characterized by an embankment slope transition at the crest to a flatter top slope as indicated in Figure 4.5b. The drainage basin that develops on the top slope is considered tributary to the gully and drainage basin that develops on the embankment slope. Therefore, the tributary drainage area at the toe of the embankment slope is the sum of the side slope length and top slope length.

$$L_t = L_o + L_2 \tag{4.13}$$

where L_t is the total length of the tributary drainage area. The drainage area for the compound slope can be estimated with Equation (4.10).

4.8.3 Type 3 Embankment

The Type 3 embankment is characterized by an embankment slope transition at the crest to an adverse slope as presented in Figure 4.5c. The gully bottom slope, S_b, evolves to be flatter than the original embankment slope, S_o. Based upon flume and field observations, the gully will extend through the crest into the cap as indicated in Figure 4.5c. The total tributary drainage area to the embankment toe is the sum of the drainage from the embankment side slope and the area tributary to the gully on the adverse slope. Based upon Figure 4.4, the gully bottom slope can be estimated as:

$$S_b = [1.008 * S_o] - 0.063 \qquad (4.9)$$

where S_b is the gully bottom slope, and S_o is the design embankment side slope, both slopes are expressed in decimal form. The effective embankment length, L_3, that compensates for the embankment side slope length, L_o, and the top slope length for the adverse slope may be computed as:

$$L_3 = 1.175 * L_o \qquad (4.14)$$

where L_o and L_3 are expressed in meters.

The tributary drainage area may be estimated as:

$$A = 0.276 * [L_3 * \cos(\theta)]^{1.636} \tag{4.15}$$

where θ is the gully bottom slope angle expressed in degrees. It should be noted that Equations (4.9), (4.14), and (4.15) may be applied to a Type 1 embankment if the gully extends into the horizontal top slope.

4.9 DESIGN PROCEDURE

Utilizing the analysis presented in the preceding sections, a comprehensive design procedure has been developed. The purpose of the procedure is to provide a design engineer the tools to predict the potential for gully intrusion into a reclaimed slope. As is the case with any empirically derived design procedure, the limits of the governing equations must be identified.

Analysis presented in the preceding sections, and studies conducted by Elliott (1990), Leopold *et al.* (1964), Pauley (1993), and Abt *et al.* (1994), have indicated that the total volume of runoff conveyed down an embankment over the course of the design life is the governing variable in estimating the potential depth of gully intrusion. Runoff volume data extracted from field studies (Falk, 1985; Abt *et al.*, 1994) ranged from approximately 3 to 3,681 m^3. Flume studies, conducted by Pauley (1993), provided total runoff volume data ranging from approximately 2,618 to 6,628 m^3. Application of the proposed design procedure requires engineering judgement and an understanding of the limits of variation within the database utilized in the analysis.

Step 1

The initial step of the gully incision estimation procedure is to determine the embankment design life. The embankment design life, N (measured in years), is defined as the period in which little or no maintenance is performed.

Step 2

Embankment geometries commonly employed have been segmented into three categories: Type 1, Type 2, or Type 3 embankment. Figures 4.5a thru 4.5c presents a schematic of the three embankment types and the appropriate design variables. The type of embankment used in the initial design will have to be determined.

A Type 1 embankment, shown in Figure 4.5a, is characterized by a uniformly sloped face that transitions to a flat top at the crest. The design variables are the original embankment slope (S_o), the original embankment height (H_o), and the original embankment length (L_o).

Type 2 embankments are characterized by an embankment slope that transitions into a flatter top slope. A schematic of a Type 2 embankment is presented in Figure 4.5b. Design variables include those described for a Type 1 embankment as well as the slope of the embankment cap (S_2), the length of the top slope (L_2), and the height of the top slope (H_2).

The characteristic geometry of a Type 3 embankment, shown in Figure 4.5c, incorporates a uniform slope that transitions into an adverse top slope at the embankment crest. Design variables required for a Type 3 embankment include those presented for a Type 1 embankment in addition to the transitional slope length (L_3).

Step 3

Abt *et al.* (1995) reports that the prominent soil parameter affecting the potential for gully intrusion into an embankment is the percent clay content of the soil. As such, three thresholds for clay content of the embankment material have been identified and correspond to clay contents less than 15%, clay contents greater than 15% but less than 50%, and clay contents greater than 50%. The composition of the embankment material in terms of percentage sands, silts and clays will need to be determined.

Step 4

The average annual precipitation (P), expressed in meters, must be determined for the embankment site. Estimates of precipitation can be obtained from U.S. Weather Bureau isohyetal maps, local climatological data or other appropriate means.

Step 5

In order to determine the volume of runoff to which an embankment will be exposed during its design life, the drainage area tributary to the embankment will need to be determined or estimated. For embankments without external drainage basins, the tributary drainage area that forms on the face of the embankment will determine the total volume of runoff (Abt *et al.*, 1995). The tributary drainage area that forms on the embankment face is a unique function of the type of embankment being evaluated and the procedures outlined in Section 4.8 must be implemented. In situations where the embankment toe is exposed to runoff that develops on a tributary drainage area external to the embankment, the supplemental area (A_x) is added to the drainage area value computed using Equation (4.10).

Step 6

Once the potential tributary drainage area for the embankment is determined, the total volume of runoff over the design life of the embankment is calculated as:

$$V_r = D_t * R_r * A \quad (4.12)$$

where: V_r = the cumulative volume of runoff (m^3) from Equation (4.12);

R_r = the runoff to rainfall ratio;

A = the tributary drainage area (m^2); and

D_t = the potential depth of precipitation (m) from Equation (4.11).

and D_t is computed as the product of the average annual precipitation (P), the precipitation adjustment coefficient (C_p), and the number of years in the design life (N) as presented in Equation (4.11).

Step 7

The gully factor, as defined by Equation (4.5), is computed for the appropriate clay content of the embankment by combining Equation (4.5) with either Equation (4.2), (4.3) or (4.4), as shown in Equations (4.16), (4.17), and (4.18).

Clay content less than 15%:

$$G_f = \frac{1}{\left[2.25 + \left(0.789 * \frac{V_r}{H_0^3}\right)^{-0.55}\right]} \qquad (4.16)$$

Clay content less than 15% but less than 50%:

$$G_f = \frac{1}{\left[2.80 + \left(0.197 * \frac{V_r}{H_0^3}\right)^{-0.70}\right]} \qquad (4.17)$$

Clay content greater than 50%:

$$G_f = \frac{1}{\left[3.55 + \left(0.076 * \frac{V_r}{H_0^3}\right)^{-0.85}\right]} \qquad (4.18)$$

where: G_f = the gully factor;

V_r = the cumulative volume of runoff (m^3); and

H_o = the original embankment height (m).

Once the gully factor is calculated, an estimate of the maximum depth of gully intrusion is determined from Equation (4.19).

$$D_{max} = L_i * S_i * G_f \tag{4.19}$$

where: D_{max} = the depth of deepest gully incision (m):

L_o = the appropriate length of Type 1, 2 or 3 embankment;

S_o = the appropriate original slope of Type 1, 2 or 3 embankment; and

G_f = the gully factor.

Step 8

Once the value of D_{max} is determined using Equation (4.5) and either Equation (4.2), (4.3) or (4.4), the top width of the gully at the deepest incision can be calculated as:

$$W = \left(\frac{D_{max}}{0.61} \right)^{1.149} \tag{4.20}$$

where: W = the top width of gully (m); and

D_{max} = the depth of deepest gully incision (m).

Step 9

The location of the deepest gully incision (D_f), in terms of the number of D_{max}'s downslope from the embankment crest, can be calculated as follows:

$$D_f = 0.713 * \left(\frac{V_r * S_o}{L_o^3} \right)^{-0.415} \qquad (4.6)$$

where: D_f = the location of D_{max};

V_r = the cumulative volume of runoff (m^3);

S_o = the original embankment slope (rise/run); and

L_o = the original embankment length (m).

4.10 STABILITY ANALYSIS

Calculating the estimated depth of maximum gully incision, the approximate location of the point of maximum gully incision, and knowing the original impoundment geometry permits an overall stability analysis of the impoundment to be conducted. An approximate profile of the gully should be superimposed on a schematic of the waste impoundment as shown in Figure 4.6. If the waste storage cell within the impoundment is breached by the gully, the design must be modified. It is recommended that the waste area be designed so that a clearance of 0.5 D_{max} exists from the point of maximum gully incision to any edge of the waste storage cell.

4.11 LIMITATIONS

The empirical relationships presented to predict the maximum depth of potential gully incision (D_{max}) and the location of the maximum depth of gully incision (D_f) were developed as dimensionless relationships capable of being applied over a broad range of geometric, hydrologic, and geologic conditions. However, until a thorough field verification can be conducted, all the equations presented in the design procedure outlined

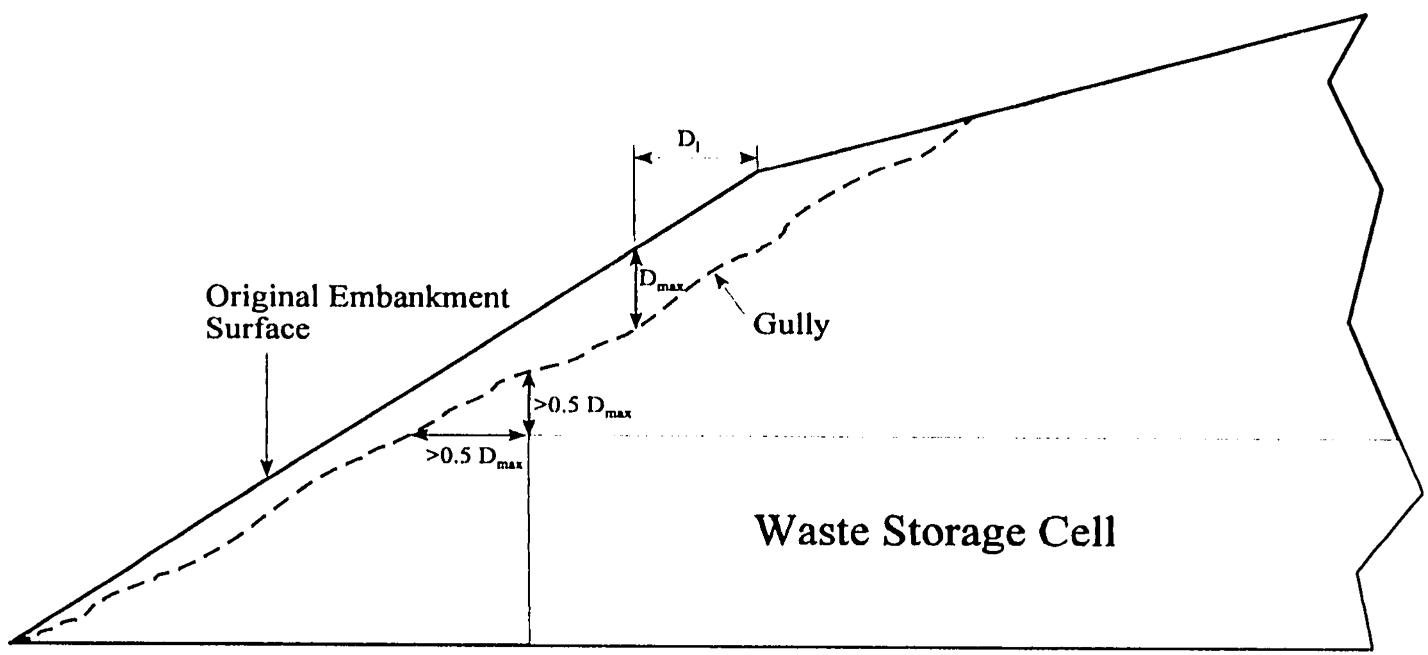

Figure 4.6 Approximate Profile of the Gully Superimposed on Waste Impoundment Schematic

in Section 4.9 should be applied only within the variation of the variables presented in Table 4.1.

Soil characteristics are the most important factors to consider when applying the design procedure. Soils used in the laboratory study, and found at the field sites, represented highly compacted soils of varying clay content native to the arid and semi-arid regions of the western United States. Application of the design procedure presented within this research to soils exhibiting highly erosive clays, in place compaction values less than approximately 50% of Standard Proctor or are of a type not representative of the arid and semi-arid regions of the western United States is left to the discretion of the user.

Application of the design procedure to embankment geometries varying significantly from the descriptions presented in Section 3.2 is left to the discretion of the user. Specifically, embankment heights greater than 175 ft have not been identified in the field and slopes greater than 70% have not been incorporated into the analysis. Field data provided data for gullies ranging in age from 5 to 35 years and flume data provided total volume of runoff data simulating a 200-year period in the arid and semi-arid regions of the western United States. Extrapolation of Equations (4.2), (4.3), and (4.4) beyond 200-year design life, or in regions exhibiting significant variation in hydrologic conditions than the arid and semi-arid regions of the western United States, is left to the discretion of the user until the design procedure can be applied and verified.

CHAPTER 5

APPLICATION EXAMPLES

The gully estimation methodology outlined in Chapter 4 will be illustrated by way of several examples. Each of the three embankment types presented in Figures 4.5a through 4.5c will be examined. It is the purpose of the examples to outline the stability assessment procedure, not to compare embankment types.

5.1 EXAMPLE ONE

5.1.1 Type 1 Embankment

As outlined in Section 4.8, a Type 1 embankment is characterized by a uniformly sloped face that transitions to a flat top at the crest. A schematic of a typical Type 1 embankment has been presented in Figure 4.5a. Through the use of the design curves presented in Figures 4.1, 4.2, 4.3 and 4.4 a Type 1 embankment will be evaluated, demonstrating the design procedure presented in Section 4.9.

5.1.2 Design Life

The embankment design life will be 200 years.

5.1.3 Embankment Geometry

Once the embankment type is determined, the initial design variables are required. The proposed embankment has the following physical dimensions:

H_o	=	embankment height	=	9 m
L_o	=	embankment slope length	=	55 m
S_o	=	embankment slope	=	0.15 rise/run

The calculations presented in Chapter 4 require that the embankment slope be expressed in degrees. The angle corresponding to a 15% slope is 8.53 degrees.

5.1.4 Soil Composition

It has been determined from a soil analysis that the embankment material is composed of 20% clay by volume. In addition, a Standard Proctor test indicated that the embankment is to be placed at 90% compaction.

5.1.5 Precipitation

Local climatological data indicate an average annual precipitation of 0.20 meters for the construction site. In addition, a value of 0.13 is determined from Gebert *et al.* (1989) as the runoff to rainfall ratio, R_r, for the area.

5.1.6 Potential Tributary Drainage Area

As presented in Equation (4.10), the tributary drainage area for a Type 1 embankment is estimated as:

$$A = 0.276 * [L_o * \cos(\theta)]^{1.636} \tag{4.10}$$

where: A = the tributary area (m^2);

L_o = the slope length (m); and

θ = the slope angle (degrees).

Substituting the geometric values determined in Section 5.1.3 for the slope length and slope angle, Equation (4.10) becomes:

$$A = 0.276 * [55 \text{ m} * \cos(8.53)] \tag{5.1}$$

or

$$A = 190.65 \text{ m}^2$$

Thus, the potential tributary drainage area at the toe of the embankment is 190.65 m^2.

5.1.7 Runoff Volume

The first step in computing the total runoff volume for the site is to determine the potential depth of precipitation, D_t, the site will be exposed to during the design life. As shown in Equation (4.11), the total depth of precipitation is the product of the average annual precipitation, the precipitation adjustment coefficient, and the design life. For the purpose of this example, it is assumed that no site specific rationale for adjusting the average annual precipitation value determined from local weather stations, state climatological databases, or from U.S. Weather Bureau isohyetal maps exists and a precipitation adjustment coefficient, C_p, of 1.0 will be used.

Substituting into Equation (4.11) yields:

$$D_t = 0.20 \text{ m} * 1.0 * 200 \text{ years} \tag{5.2}$$

or

$$D_t = 40.0 \text{ m of precipitation}$$

and a potential depth of precipitation of 40.0 m is computed. The cumulative volume of runoff, V_r, is defined as the product of the potential depth of precipitation, D_t, the runoff to rainfall ratio, R_r, and the potential tributary drainage area, A, as presented in Equation (4.12). Substituting the values of D_t, R_r and A obtained above into Equation (4.12) yields:

$$V_r = 40.0 \text{ m} * 0.13 * 190.65 \text{ m}^2 \tag{5.3}$$

or

$$V_r = 991.38 \text{ m}^3$$

Therefore, the embankment toe will drain 991.38 m^3 of runoff during the 200-year design life.

5.1.8 Maximum Depth of Gully Incision

The gully factor, G_f, for the embankment must be determined in Section 4.2.1. A clay content of 20 % in the embankment material requires that Equation (4.17) be used to calculate the gully factor. Substituting values for H_o and V_r into Equation (4.17) yields:

$$G_f = \frac{1}{2.80 + \left[0.197 * \left\{\frac{991.38 \text{ m}^3}{(9.0 \text{ m})^3}\right\}\right]^{-0.70}} \tag{5.4}$$

or

$$G_f = 0.188$$

A gully factor of 0.188 is entered into Equation (4.19) to determine the maximum depth of gully incision as follows:

$$D_{max} = 0.188 * 55.0 \text{ m} * 0.15 \tag{5.5}$$

or

$$D_{max} = 1.55 \text{ m}$$

Thus, after a 200-year period, a gully incision 1.55-m deep would be expected on the face of the embankment.

5.1.9 Location of Maximum Depth

Equation (4.6) presents an empirical relation predicting the location of D_{max} as a function of the total volume of runoff, embankment length, and embankment slope. Substituting the values determined above into Equation 4.6 yields

$$D_{\ell} = 0.713 * \left\{ \frac{(991.38 \text{ m}^3 * 0.15)}{(55 \text{ m})^3} \right\}^{-0.415} \tag{5.6}$$

or

$$D_{\ell} = 13.16$$

which represents the number of D_{max}'s downslope from the crest the deepest incision is expected to occur. To determine the location in meters, multiply the value determined for D_{ℓ} by that calculated for D_{max}. For this example, the deepest incision point will occur approximately 20.4 m downslope from the embankment crest.

5.1.10 Gully Top Width

Equation (4.20) presents an empirical relationship that can be used to predict gully top width, W, as a function of maximum gully incision, D_{max}. Substituting the value of 1.55 meters computed for D_{max} into Equation (4.20):

$$W = \left(\frac{1.55\ m}{0.61} \right)^{1.149} \tag{5.7}$$

or

$$W = 2.92\ m$$

Therefore, 2.92 m would be the estimated gully width at the point of deepest gully incision. Summarizing the results obtained above yields:

$D_{max} = 1.55$ m;

$D_t = 20.4$ m; and

$W = 2.92$ m.

The results of the embankment stability analysis should be compared to the design geometry, as described in Section 4.10, to ensure that the gully does not incise into the top of the waste.

5.2 EXAMPLE TWO

5.2.1 Type 2 Embankment

Type 2 embankments, presented in Figure 4.5b, are identified by an embankment slope that transitions into a flatter top slope. Embankments constructed with Type 2 geometry are evaluated by superimposing two Type 1 embankments. The potential tributary drainage area that develops due to the top slope is added to the tributary drainage area at the toe of the embankment face. The following example is used to outline the procedure of stability analysis of a Type 2 embankment.

5.2.2 Design Life

An embankment design life of 200 years will be evaluated.

5.2.3 Embankment Geometry

Once the embankment type is determined, the initial design variables are required. It will be assumed that the embankment has the following physical dimensions, corresponding to the variables identified in Figure 4.5b.

H_o	=	embankment height	=	9 m
L_o	=	embankment slope length	=	55 m
S_o	=	embankment slope	=	0.15 rise/run
H_2	=	top embankment height	=	5 m
L_2	=	top embankment length	=	100 m
S_2	=	top embankment slope	=	0.05 rise/run

The calculations presented in Chapter 4 require that the embankment slope be expressed in degrees. The angles corresponding to a 15 and 5% slope are 8.53 and 2.86 degrees, respectively.

5.2.4 Soil Composition

It has been determined from a soil analysis that the embankment material is composed of 13% clay by volume. In addition, a Standard Proctor test indicated that the embankment is to be placed at 90% compaction.

5.2.5 Precipitation

Local climatological data indicate an average annual precipitation of 0.20 meters for the construction site. In addition, a value of 0.13 is determined from Gebert *et al.* (1989) as the runoff to rainfall ratio, R_r, for the area.

5.2.6 Potential Tributary Drainage Area

The total potential tributary drainage area for a Type 2 embankment is determined by summing the drainage area calculated by applying Equation (4.10) to each of the two embankment faces, as shown below:

$$A_t = A_1 + A_2 \tag{5.8}$$

where: A_t = the total tributary drainage area (m^2);

A_1 = the tributary drainage area of lower embankment face (m^2); and

A_2 = the tributary drainage area of top embankment face (m^2).

Substituting the design variables provided in Section 5.2.3 into Equations (4.10) and (5.8) for each embankment face gives:

Side Slope:

$$A_1 = 0.276 * [55 \text{ m} * \cos(8.53)]^{1.636} \tag{5.9}$$

or

$$A_1 = 190.65 \text{ m}^2$$

Top Slope:

$$A_2 = 0.276 * [100 \text{ m} * \cos(2.86)]^{1.636} \tag{5.10}$$

or

$$A_2 = 515.25 \text{ m}^2$$

The values determined for A_1 and A_2 are then used to compute A_t, as shown below:

$$A_t = 190.65 \text{ m}^2 + 515.25 \text{ m}^2 \tag{5.11}$$

or

$$A_t = 705.90 \text{ m}^2$$

Therefore, the total potential tributary drainage area for the Type 2 embankment is 705.90 m^2.

5.2.7 Runoff Volume

The first step in computing the total runoff volume for the site is to determine the potential depth of precipitation, D_t, the site will be exposed to during the design life. As shown in Equation (4.11), the total depth of precipitation is the product of the average annual precipitation, the precipitation adjustment coefficient, and the design life. For the purposes of this example, it is assumed that no site specific rationale for adjusting the average annual precipitation value determined from local weather stations, state climatological databases or from U.S. Weather Bureau isohyetal maps exists and a precipitation adjustment coefficient, C_p, of 1.0 will be used. Substituting into Equation (4.11) yields:

$$D_t = 0.20 \text{ m} * 1.0 * 200 \text{ years} \tag{5.12}$$

or

$$D_t = 40.0 \text{ m of precipitation}$$

and a potential depth of precipitation of 40.0 m is computed. The cumulative volume of runoff, V_r, is defined as the product of the potential depth of precipitation, D_t, the runoff to rainfall ratio, R_r, and the potential tributary area, A. Substituting the values of D_t, R_r, and A_t obtained above into Equation (4.12) yields

$$V_r = 40.0 \text{ m} * 0.13 * 705.90 \text{ m}^2 \tag{5.13}$$

or

$$V_r = 3{,}670.68 \text{ m}^3$$

Therefore, the embankment toe will drain 3,670.68 m^3 of runoff during the 200-year design life.

5.2.8 Maximum Depth of Gully Incision

The gully factor, G_f, for the embankment must be determined as outlined in Section 4.2.1. A clay content of 13% in the embankment material requires that Equation (4.16) be used to calculate the gully factor. Substituting values for H_o and V_r into Equation (4.16) yields:

$$G_f = \frac{1}{2.25 + \left[0.789 * \left\{\frac{3{,}670.78 \text{ m}^3}{(9.0 \text{m})^3}\right\}\right]^{-0.55}} \tag{5.14}$$

or

$$G_f = 0.368$$

A gully factor of 0.368 is entered into Equation (4.19) to determine the maximum depth of gully incision as follows:

$$D_{max} = 0.368 * 55.0 \text{ m} * 0.15 \tag{5.15}$$

or

$$D_{max} = 3.04 \text{ m}$$

Thus, after a 200-year period, a gully incision 3.04-m deep would be expected on the face of the embankment.

5.2.9 Location of Maximum Depth

Equation (4.6) presents an empirical relation predicting the location of D_{max} as a function of the total volume of runoff, embankment length, and embankment slope. Substituting the values determined above into Equation (4.6) gives:

$$D_f = 0.71 * \left\{ \frac{(3{,}670.78 \text{ m}^3 * 0.15)}{(55 \text{ m})^3} \right\}^{-0.415} \tag{5.16}$$

or

$$D_f = 7.63$$

which represents the number of D_{max}'s down slope from the crest the deepest incision is expected to occur. To determine the location in meters, multiply the value determined for D_f by that determined for D_{max}. For this example the deepest incision point will occur approximately 23.2 meters down slope from the embankment crest.

5.2.10 Gully Top Width

Equation (4.20) presents an empirical relationship that can be used to predict gully top width, W, as a function of maximum gully incision, D_{max}. Substituting the value of 3.04 m computed for D_{max} into Equation (4.20) gives:

$$W = \left(\frac{3.04 \ \text{m}}{0.61} \right)^{1.149} \tag{5.17}$$

or

$$W = 6.33 \text{ m}$$

Therefore, 6.33 m would be the estimated gully width at the point of deepest gully incision. Summarizing the results obtained above yields:

$$D_{max} = 3.04 \text{ m;}$$

$$D_c = 23.8 \text{ m; and}$$

$$W = 6.33 \text{ m.}$$

The results of the embankment stability analysis should be compared to the design geometry, as described in Section 4.10, to ensure that the gully does not incise into the top of the waste.

5.3 EXAMPLE THREE

5.3.1 Type 3 Embankment

Type 3 embankments, shown in Figure 4.5c, are characterized by a main embankment face that transitions into an adverse top slope. Type 3 embankments are analyzed similarly to Type 1 embankments. The only difference in the calculations, is an effective embankment length, L_3, must be determined as outlined in Section 4.8.3.

5.3.2 Design Life

An embankment design life of 1,000 years will be evaluated.

5.3.3 Embankment Geometry

The following design variables will be assumed for the Type 3 embankment:

H_o = embankment height = 9 m

L_o = embankment slope length = 55 m

S_o = embankment slope = 0.15 rise/run

The calculations presented in Chapter 4 require that the embankment slope be expressed in degrees. The angle corresponding to a 15% slope is 8.53 degrees.

5.3.4 Soil Composition

It has been determined from a soil analysis that the embankment material is composed 55% clay by volume. In addition, a Standard Proctor test indicated that the embankment was to be placed at 90% compaction.

5.3.5 Precipitation

Local climatological data indicate an average annual precipitation of 0.20 m for the construction site. In addition, a value of 0.13 is determined from Gebert *et al.* (1989) for the runoff to rainfall ratio, R_r, for the area.

5.3.6 Potential Tributary Drainage Area

The geometry of a Type 3 embankment is such that the gully could extend past the embankment crest and onto the adverse slope, thus increasing the effective length of the embankment. Flume and field observations have indicated that the effective length of a Type 3 embankment may be approximated from Equation (4.14). Substituting the value assumed above for L_0 into Equation (4.14) gives:

$$L_3 = 1.175 * 55 \text{ m} \tag{5.18}$$

or

$$L_3 = 64.63 \text{ m}$$

The effective embankment length calculated above is then entered into Equation (4.15) as follows:

$$A = 0.276 * [64.63 \text{ m} * \cos(8.53)]^{1.636} \tag{5.19}$$

or

$$A = 248.24 \text{ m}^2$$

Therefore, the total potential tributary drainage area at the toe of the embankment is 248.24 m^2.

5.3.7 Runoff Volume

The first step in computing the total runoff volume for the site is to determine the potential depth of precipitation, D_t, the site will be exposed to during the design life. As shown in Equation (4.11), the total depth of precipitation is the product of the average annual precipitation, the precipitation adjustment coefficient, and the design life. For the purposes of this example, it is assumed that no site specific rationale for adjusting the average annual precipitation value determined from local weather stations, state climatological databases or from U.S. Weather Bureau isohyetal maps exists and a precipitation adjustment coefficient, C_p, of 1.0 will be used. Substituting into Equation (4.11) yields:

$$D_t = 0.20 \text{ m} * 1.0 * 1{,}000 \text{ years} \quad (5.20)$$

or

$$D_t = 200.0 \text{ m of precipitation}$$

and a potential depth of precipitation of 200.0 m is computed. The cumulative volume of runoff, V_r, is defined as the product of the potential depth of precipitation, D_t, the runoff to rainfall ratio, R_r, and the potential tributary area, A. Substituting the values of D_t, R_r, and A obtained above into Equation (4.12) yields:

$$V_r = 200.0 \text{ m} * 0.13 * 248.24 \text{ m}^2 \quad (5.21)$$

or

$$V_r = 6{,}454.24 \text{ m}^3$$

Therefore, the embankment toe will drain 6,454.24 m^3 of runoff during the 1,000-year design life.

5.3.8 Maximum Depth of Gully Incision

The gully factor, G_f, for the embankment must be determined as outlined in Section 4.8.1. A clay content of 55 % in the embankment material requires that Equation (4.18) be used to calculate the gully factor. Substituting values for H_o and V_r into Equation (4.18):

$$G_f = \frac{1}{3.55 + \left[0.076 * \left\{\frac{6{,}454.2\ m^3}{(9.0\ m)^3}\right\}\right]^{-0.85}} \tag{5.22}$$

or

$$G_f = 0.202$$

A gully factor of 0.202 is entered into Equation (4.19) to determine the maximum depth of gully incision as follows:

$$D_{max} = 0.202 * 55.0\ m * 0.15 \tag{5.23}$$

or

$$D_{max} = 1.66\ m$$

Thus, after a 1,000-year period, a gully incision 1.66-m deep would be expected on the face of the embankment.

5.3.9 Location of Maximum Depth

Equation (4.6) presents an empirical relation predicting the location of D_{max} as a function of the total volume of runoff, embankment length, and embankment slope. Substituting the values determined above into Equation (4.6) gives:

$$D_{\ell} = 0.713 * \left\{\frac{(6{,}454.2\ m^3 * 0.15)}{(55\ m)^3}\right\}^{-0.415} \tag{5.24}$$

or

$$D_{\ell} = 6.04$$

which represents the number of D_{max}'s downslope from the crest the deepest incision will occur. To determine the location in meters, multiply the value determined for D_{ℓ} by that determined for D_{max}. For this example the deepest incision point will occur 10.03 m downslope from the embankment crest.

5.3.10 Gully Top Width

Equation (4.20) presents an empirical relationship that can be used to predict gully top width, W, as a function of maximum gully incision, D_{max}. Substituting the value of 1.66 m computed for D_{max} into Equation (4.20) yields:

$$W = \left(\frac{1.66 \text{ m}}{0.61} \right)^{1.149} \tag{5.25}$$

or

$$W = 3.16 \text{ m}$$

Therefore, 3.16 m would be the estimated gully width at the point of deepest gully incision. Summarizing the results obtained above yields:

$$D_{max} = 1.66 \text{ m};$$

$$D_{\ell} = 6.04 \text{ m; and}$$

$$W = 3.16 \text{ m}.$$

The results of the embankment stability analysis should be compared to the design geometry, to ensure that the gully does not incise into the top of the waste.

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

6.1 SUMMARY

The long-term integrity and stability of nuclear low-level waste and uranium tailings impoundments has been congressionally mandated in the Surface Mining Control and Reclamation Act of 1977, Public Law 95-87, the Uranium Mill Tailings Radiation Control Act of 1978, Public Law 95-604, and in regulation 40 CFR 192. Stabilization is to be effective for periods of 200 to 1,000 years, to protect the health and safety of the public.

The impoundments constructed to encapsulate the waste materials must resist the natural erosive processes to prevent exposure and release of the waste. One erosive process that has been difficult to predict is gully intrusion. This study was conducted to analyze the gully erosion processes and determine its effect on long-term embankment stability and estimate the potential impacts on waste stabilization. The investigation was conducted in four distinct phases: (1) literature search and background information collection; (2) performance of a laboratory simulation study; (3) field data collection; and (4) data compilation, analysis, and synthesis.

A comprehensive procedure has been presented for estimating the magnitude and location of a potential gully intrusion into a soil covered, waste impoundment. The estimation procedure requires that the user obtain information pertaining to the regional hydrology, site soils, proposed impoundment geometry, and design life. It is noted that while a limited set of field data was available for synthesis into the prediction equations, the procedure presented is a first step into the determination of the magnitude and location of gullying into sloped surfaces.

6.2 FINDINGS

- A database was assembled consisting of 27 field sites and 7 laboratory experiments. The geometric, hydrologic, and geologic variables presented in Table 3.1 were documented for each embankment included in the database.
- Three dimensionless equations were presented to estimate the maximum depth of gully incision (D_{max}) as a function of the cumulative volume of runoff (V_r), the embankment height, (H_o), embankment slope length (L_o), the embankment slope (S), and the clay content of the soil matrix comprising the embankment. As presented in Equations (4.2), (4.3) and (4.4), the derived relationships are as follows:

$$\frac{D_{max}}{L_o * S_o} = \frac{1}{2.25 + \left(0.789 * \frac{V_r}{H_o^3}\right)^{-0.55}} \tag{4.2}$$

$$\frac{D_{max}}{L_o * S_o} = \frac{1}{2.80 + \left(0.197 * \frac{V_r}{H_o^3}\right)^{-0.70}} \tag{4.3}$$

$$\frac{D_{max}}{L_o * S_o} = \frac{1}{3.55 + \left(0.76 * \frac{V_r}{H_o^3}\right)^{-0.85}} \tag{4.4}$$

- A dimensionless expression was derived to correlate the location of the deepest point of gully incision (D_ℓ), measured downslope from the crest, to a ratio of the cumulative runoff volume, slope angle and slope length. The location of the maximum depth of incision may be estimated using Equation (4.6) by:

$$D_\ell = 0.713 * \left(\frac{(V_r * S_o)}{L_o^3}\right)^{-0.415} \tag{4.6}$$

- Computations of potential sediment transport volumes out of a gully require an estimation of the gully width. Two equations were developed to predict the width of a gully as a function of the depth of maximum gully incision. A best-fit and envelope power regression was conducted to correlate the depth of maximum gully incision to an estimate of the gully width at the location of that depth. Equations (4.7) and (4.8) presented the best-fit and envelope expressions, respectively, as:

$$W = \sqrt[0.87]{\frac{D_{max}}{0.61}} \tag{4.7}$$

$$W = \sqrt[0.7]{\frac{D_{max}}{1.60}} \tag{4.8}$$

- Analysis of the flume data indicated that as the total volume of runoff conveyed through the flume increased, the slope of the gully bottom (S_b) flattened. Combining the flume and field data and regressing the gully bottom slope against the original embankment slope (S_o), it was observed that the gully bottom slope flattened approximately 15%. Equation (4.9) was presented as an expression to estimate the gully bottom slope as:

$$S_b = [1.008 * S_o] - 0.063 \tag{4.9}$$

- A long-term, time averaged design procedure was presented as a tool to estimate the effects of gully intrusion into a reclaimed slope. The step by step procedure was illustrated by way of several examples utilizing impoundment geometries identified in the field.

6.3 RECOMMENDATIONS FOR FUTURE RESEARCH

The following items have been identified as areas of future research for examining the long-term, time averaged effects of gully erosion on embankment slopes:

- Apply the proposed design procedure to a field location over time and document the predicted versus observed values for maximum depth of gully incision, location of the depth of maximum incision, and the width at the location of maximum incision.
- Conduct full scale laboratory tests to provide 1,000 years of total runoff volumes. Data used in the analysis simulated approximately 200 years of runoff and verification of the extrapolation of design life is warranted.

- Identify field sites to complement the database. While it is uncommon to find reclaimed waste impoundment with gullies older than 10 to 20 years, other sources of data do exist. Highway embankments are similar in geometry to waste impoundments and could provide a supplemental data set with considerable variability. An investigation into the possibility of incorporating highway embankment data could only strengthen the analysis presented.
- A risk analysis of the variables and expressions used in the development of the design procedure should be conducted. When a design requires a 1,000-year design life, an understanding of the risk associated with each assumption would provide useful insight for engineering judgement.
- Conduct laboratory experiments to investigate the effects of additional soil types. While the procedure presented is a function of clay content, variables such as compaction, Plasticity Index, and grain size have been identified as significant in the gully formation process.
- The presence of vegetation on an embankment should be investigated to allow a "credit" to be taken for the added stability caused by established vegetation. Due to the limited number of field sites, an attempt to quantify the effects of vegetation in this study was unsuccessful.

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APPENDIX A

SITE DESCRIPTIONS

Gully G-1

Gully G-1 was located in Colorado on the ridge of an abandoned mine pit. The drainage area tributary to the gully was approximately 33,541 ft^2. Over a period of approximately 12 years, the gully had incised to a maximum depth of 7.2 ft with a width at the location of maximum depth of 15 ft. Soils comprising the 203-ft long embankment were quite erodible and consisted primarily of sands. Vegetative cover on the embankment was estimated to be approximately 25%. Figure A.1 presents a photograph of Gully G-1.

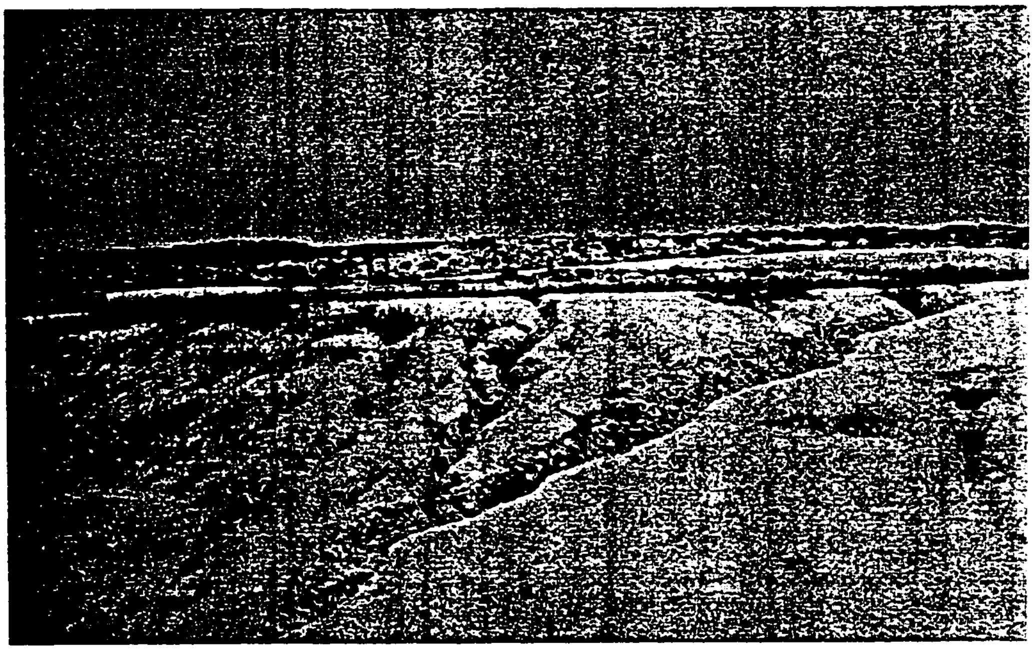

Figure A.1 Photograph of Gully G-1

Gully G-2

Gully G-2 was located in northwestern Colorado on the face of a uranium mine spoil pile. The gully had incised into the pile over a period of approximately 30 years and had reached a length of 336 ft. The gully had a maximum incision of 13.5 ft with a corresponding width of 16.5 ft. The entire area atop of the pile, approximately 40,511 ft^2, drained into the gully. The soil was primarily composed of sands and was considered highly erodible. Figure A.2 presents a photograph of Gully G-2.

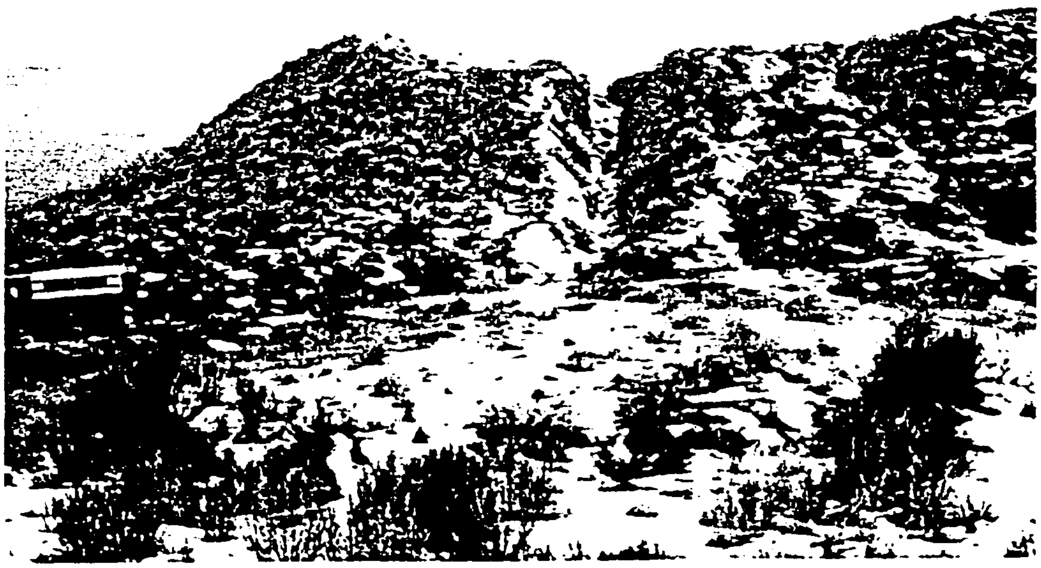

Figure A.2 Photograph of Gully G-2

Gully G-3

Gully G-3, located in northeastern Wyoming, was located on the banks of a reclaimed bentonite mine. The documented gully, with a maximum depth of 2.4 ft, a length of 292 ft and a width of 2.2 ft, was located on an embankment with an approximately 3% slope. A tributary drainage area of approximately 1,965,000 ft^2 supplied runoff to the embankment. Vegetative cover was estimated to be approximately 60%. Figure A.3 presents a photograph of Gully G-3.

Figure A.3 Photograph of Gully G-3

Gully G-4

Gully G-4, located in northeastern Wyoming, was located on the overburden pile of a reclaimed coal mine. A tributary drainage area of approximately 15,753 ft had produced a gully on the overburden pile 7.3-ft deep, 18-ft wide and 165-ft long in a 5-year period. Vegetative cover on the overburden pile was estimated to be less than 5%. Figure A.4 presents a photograph of Gully G-4.

Figure A.4 Photograph of Gully G-4

Gully G-5

Gully G-5 was located in northeastern Wyoming on the overburden pile of a reclaimed coal mine. Over a 4-year period, a tributary drainage area of approximately 15,983 ft^2 channeled runoff to form a gully 101-ft long, 2.7-ft deep, and 9.5-ft wide at the location of maximum depth. A vegetative cover of approximately 10% was estimated on the cover composed primarily of clay and slate materials. Figure A.5 presents a photograph of Gully G-5.

Figure A.5 Photograph of Gully G-5

Gully G-6

Gully G-6 was located on a bentonite spoil pile in northeastern Wyoming. Over a 4.5-year period, a gully 1.2-ft deep, 60-ft long, and 3.0-ft wide formed on an embankment face laid at a slope of 13%. A drainage area of approximately 3,944 ft^2 was tributary to the gully. Soil composed of clay and slate supported a vegetative cover of approximately 40%. Figure A.6 presents a photograph of Gully G-6.

Figure A.6 Photograph of Gully G-6

Gully G-7

Gully G-7, located in northwestern Colorado, was located on the overburden pile of reclaimed coal mine. In an 8-year period, the gully had incised to a maximum depth of 1.5 ft with a corresponding width of 3.0 ft. Runoff was produced from an approximately 191,740 ft^2 tributary drainage area. The 11% slope supported a vegetative cover of approximately 50%. Figure A.7 presents a photograph of Gully G-7.

Figure A.7 Photograph of Gully G-7

Gully G-8

Gully G-8, located in northwestern Colorado, was located on the overburden pile of reclaimed coal mine. The 190-ft long gully had incised to a maximum depth of 2.7 ft with a corresponding width of 2.5 ft. Runoff was produced from an approximately 51,166 ft^2 tributary drainage area. The 22% slope was comprised of fine sands, silt and slay material and supported a vegetative cover of approximately 50%. Figure A.8 presents a photograph of Gully G-8.

Figure A.8. Photograph of Gully G-8

Gully G-9

Gully G-9, located in northern central Wyoming, was located on the slope of an abandoned coal mine pit. In a 5-year period, the 110-ft long gully had incised to a maximum depth of 6.5 ft with a corresponding width of 5.0 ft. Runoff was produced from an approximately 649,994 ft^2 tributary drainage area. Soils consisted of primarily sand and silt material and supported a vegetative cover of approximately 20%. Figure A.9 presents a photograph of Gully G-9.

Figure A.9 Photograph of Gully G-9

Gully G-10

Gully G-10, located in central Wyoming, was located on the reclaimed slope of an open pit uranium mine. Over a 14-year period, the 430-ft long gully had incised to a maximum depth of 8.7 ft with a corresponding width of 10.5 ft. Runoff was produced from an approximately 46,415 ft^2 tributary drainage area. The 45% slope was comprised of fine sands, silt and slay material and supported a vegetative cover of approximately 15%. Figure A.10 presents a photograph of Gully G-10.

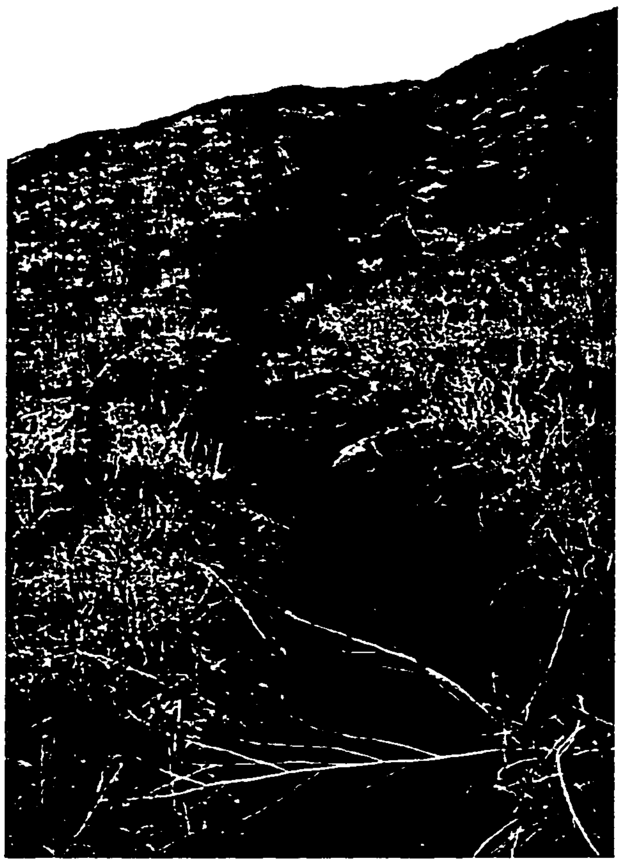

Figure A.10 Photograph of Gully G-10

Gully G-11

Gully G-11, located in southwestern Colorado, was located on the overburden pile of reclaimed coal mine. The 188-ft long gully had incised to a maximum depth of 2.8 ft with a corresponding width of 6.0 ft over a 19-year period. Runoff was produced from an approximately 5,156 ft^2 tributary drainage area. The 70% slope was armored with rocks and hard pan materials and supported a vegetative cover of approximately 5%. Figure A.11 presents a photograph of Gully G-11.

Figure A.11 Photograph of Gully G-11