GUIDELINES FOR CONSTRUCTING NATURAL GAS AND LIQUID HYDROCARBON PIPELINES THROUGH AREAS PRONE TO LANDSLIDE AND SUBSIDENCE HAZARDS

FINAL REPORT

Prepared for the

Design, Materials, and Construction Committee

of

Pipeline Research Council International, Inc.

Prepared by:

C-CORE
D.G. Honegger Consulting
SSD, Inc.

January 2009

This report is furnished to the Pipeline Research Council International, Inc. (PRCI) as a final deliverable for PRCI project ENV-1, "Pipeline Integrity Management for Geohazards." The report is the product of tasks carried out by ENV-1 Contractors; C-CORE, D.G. Honegger Consulting, and SSD, Inc.

The contents of this report are published as received from the Contractors. The opinions, findings, and conclusions expressed in the report are those of the authors and not necessarily those of PRCI, its member companies, or their representatives. Publication and dissemination of this report by PRCI should not be considered an endorsement by PRCI or the Contractors, or the accuracy or validity of any opinions, findings, or conclusions expressed herein.

The material presented in this guideline has been prepared in accordance with the standard of care for technical engineering reports. The recommendations and processes in this guideline should not be adopted without first securing competent advice with respect to the suitability for a specific application.

In publishing this report, PRCI makes no warranty or representation, expressed or implied, with respect to the accuracy, completeness, usefulness, or fitness for purpose of the information contained herein, or that the use of any information, method, process, or apparatus disclosed in this report may not infringe on privately owned rights. PRCI assumes no liability with respect to the use of, or for damages resulting from the use of, any information, method, process, or apparatus disclosed in this report.

The text of this publication, or any part thereof, may not be reproduced or transmitted in any form by any means, electronic or mechanical, including photocopying, recording, storage in an information retrieval system, or otherwise, without the prior written approval of PRCI."

Pipeline Research
Council International
LEADING PIPELINE RESEARCH

Acknowledgements

This report was prepared for PRCI as a deliverable under a project with the U.S. Department of Transportation Pipeline Hazardous Materials Safety Administration (PHMSA). In addition to the contributions by the three contractors, C-CORE, D.G. Honegger Consulting, and SSD, Inc., this report benefited from the contributions of several individuals and organizations.

The most significant contribution was provided by Rex L. Baum, Devin L. Galloway, and Edwin L. Harp at the U.S Geological Survey (USGS). Under a cooperative research and development agreement, the USGS provided a summary of the state-of-practice on landslide and subsidence hazard identification and published their summary as Open-File Report 2008-1164. Much of the material in this document related to hazard definition was extracted from the USGS Open-File Report. Dr. James Merritt, the PHMSA Organization Program Coordinator for this project, participated in the initial project meetings and provided suggestions that assisted in defining level of detail and areas of emphasis for this document.

D.G. Honegger Consulting was responsible preparation of this report, with input from C-CORE and SSD, Inc., for review by the other PRCI Contractors on this project. This effort was assisted by the suggestions and input provided by the following individuals with expertise in geotechnical engineering, landslide hazard evaluation and risk assessment, pipeline geohazards risk assessment, and techniques and methods for general monitoring of pipeline alignments and location-specific landslide monitoring:

Mr. Raymond Boivin, Plateau Engineering Ltd.

Mr. Richard Butler, Golder Associates Ltd.

Mr. Moness Rizkalla, Visitless Integrity Assessment Ltd.

An ad hoc steering group for this project was established by PRCI. This steering group provided technical review and PRCI approval of the final report. Members of the ad hoc steering group are provided below:

Richard Gailing, Steering Group Chair
Southern California Gas Company

Scott Baumgardner
Marathon Pipe Line LLC

Roger Bergman
Chevron Texaco E&P Technology Company

Guillaume L'Henoret
Gaz de France

Dave Katz
Williams Companies

Joe Zhou
TransCanada PipeLines Ltd.

The project is indebted to the following individuals contracted to serve as expert reviewers during the development of these guidelines. Their comments and suggestions were valuable in refining the content and organization of these guidelines.

Dr. Jeffrey R. Keaton, MACTEC Engineering and Consulting, Inc.

Mr. Mark E. Schmoll, URS Corporation

Table of Contents

List of Tables

List of Figures

Acronyms

ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society of Testing and Materials
DEM	digital elevation model
DInSAR	differential interferometric synthetic aperture radar
EPS	expanded polystyrene
GCP	ground control points
GIS	geographical information system
GPS	global positioning system
HDD	horizontal directional drill
InSAR	interferometric synthetic aperture radar
lidar	light detection and ranging
SAR	synthetic aperture radar
USGS	United States Geological Survey
XPS	extruded polystyrene

Nomenclature

To the extent possible, the nomenclature in these guidelines matches the nomenclature in equations from various reference sources. In several cases, this leads to duplication in nomenclature. Since many terms are well established in various technical disciplines, the potential for confusion in the use of unfamiliar terms is considered a more significant drawback than duplication.

A	amplitude of back-scattered radiation
c	cohesion representative of the soil backfill
c'	effective cohesive strength of slope material for landslide assessment
D	outside pipe diameter
f	pipe coating factor for estimating interface friction angle from internal friction angle
g	acceleration due to gravity
H	depth from the ground surface to the centerline of a buried pipeline
K_o	coefficient of earth pressure at rest
L_{anchor}	length of burial sufficient to develop yield in the pipe under relative axial soil displacement
N_c	soil bearing capacity cohesion factor for vertically downward loading in clay
N_{ch}	soil bearing capacity factor for horizontal loading in clay
N_{cv}	soil bearing capacity factor for vertically upward loading in clay
N_q	soil bearing capacity surcharge factor for vertically downward loading in sand
N_{qh}	soil bearing capacity factor for horizontal loading in sand
N_{qv}	soil bearing capacity factor for vertically upward loading in sand
N_γ	soil bearing capacity soil weight factor for vertically downward loading in sand
p	internal pipeline pressure
p_y	internal pipeline pressure that produces a hoop stress equal to σ_y
P_a	reference pressure (100 kPa ≈ 14.5 psi ≈ 1 tsf)
P_u	maximum lateral soil load caused by pipe lateral movement relative to the surrounding soil
P_y	axial load in a pipeline corresponding to a uniform tensile stress equal to the SMYS
Q_d	maximum lateral soil load caused by pipe vertically downward movement

	relative to the surrounding soil
Q_u	maximum lateral soil load caused by pipe vertically upward movement relative to the surrounding soil
t	pipe wall thickness
T_u	maximum soil force on pipeline from relative axial movement
α	adhesion factor for estimating axial pipeline soil load
δ	soil interface friction angle
Δ_p	relative displacement between pipe and soil in lateral direction necessary to develop P_u
Δ_{qd}	relative displacement between pipe and soil in vertically downward direction necessary to develop Q_d
Δ_{qu}	relative displacement between pipe and soil in vertically upward direction necessary to develop Q_u
Δ_t	relative displacement between pipe and soil in axial direction necessary to develop T_u
ϕ	soil internal friction angle
ϕ	phase of back-scattered radiation
ϕ'	effective angle of internal friction for slope stability assessment
Φ	standard normal probability function
γ	total unit weight of soil
$\bar{\gamma}$	effective (or submerged) unit weight of soil
γ_w	unit weight of water
λ	ratio of pore pressure to overburden stress
λ	half radar wavelength
σ	standard deviation
σ_y	pipe material yield stress
σ_{vo}	total soil overburden pressure
σ'_{vo}	effective soil overburden pressure

1.0 INTRODUCTION TO THE GUIDELINES

These guidelines provide recommendations for the assessment of new and existing natural gas and liquid hydrocarbon pipelines subjected to potential ground displacements resulting from landslides and subsidence. The process of defining landslide and subsidence hazards is highly dependent upon the judgments of experienced and knowledgeable specialists in geology and geotechnical engineering. With the heavy reliance on judgment, it is not possible to identify specific processes that constitute generally acceptable approaches. Therefore, much of this document focuses on identifying the variety of available methods that can be used to define landslide and subsidence hazards for pipelines.

These guidelines are intended to provide recommendations on engineering practices that result in levels of pipeline integrity for geohazards that are commensurate with the severity of the landslide or subsidence hazard and the risks (safety, operational, economic, and public welfare) posed by the hazard. Key topics that need to be covered when providing such guidance include the following:

- Identifying areas where landslide or subsidence hazards are possible
- Defining the extent, type, magnitude and severity of the potential ground displacement along the pipeline route
- Determining the breadth and scope of geological and geotechnical investigations
- Accounting for physical and environmental conditions, including potential future considerations (e.g., unusual rainfall, changes in land use, site development, flooding) that can alter the likelihood or severity of a potential geohazards
- Identifying approaches for assessing pipeline response to expected ground movements
- Identifying appropriate risk mitigation measures and evaluating their effectiveness

The term “landslide” is used in these guidelines to refer to any lateral and downward soil slope movement, regardless of the rate of movement, amount of movement, or volume of movement. In general, no differentiation is provided among landslides that are actively moving, exhibit evidence of recent movement, or judged to have a potential for future movement based upon similar characteristics of active or recent landslides. Such differentiation is really only a matter of the annual likelihood of occurrence that may be assigned to a particular landslide.

The term “subsidence” is used in these guidelines to refer to any depression of the ground surface, regardless of the causal mechanism, rate of movement, or regional extent of the depression.

These guidelines are intended to be of interest to various parties involved in the operations and regulation of natural gas and liquid hydrocarbon pipelines:

- Pipeline owners and the technical specialists that work with them to develop reasonable measures to assure adequate levels of integrity for buried pipelines through areas of landslide and subsidence hazards,
- Pipeline regulators that approve the construction and operation of pipelines

- Individuals and non-government organizations with an interest public safety and environmental impact issues associated with pipeline construction and operation.

One of the main goals of these guidelines is to provide a mechanism for improving the understanding of the types and quality of hazard information that can reasonably be provided and practical measures available to reduce or manage the risk of operating a pipeline in an area of landslide and subsidence hazards.

1.1 Scope

With very rare exceptions, buried pipelines are immune to damage from rock falls or debris deposited on the ground surface. Therefore, these guidelines only address soil movements that intersect the pipeline alignment. It should be noted that surface facilities, such as stations, meters, and valves, are exposed to hazards related to rock and debris accumulating on the alignment.

In terms of hazard definition, the potential vulnerability of buried pipelines to ground movements from landslides and subsidence only requires estimates of the length of pipeline contained within a zone of ground displacement, the variation in direction of ground displacement relative to the pipeline alignment, and the variation in the amount of displacement along the pipeline alignment. Therefore, recommendations related to categorization of landslide types and process (e.g., Varnes, 1978) or a system for classification of landslides (e.g., Turner and Schuster, 1996), that are often of great interest to geologists, are not part of these guidelines.

These guidelines do not provide explicit procedures for analytical assessment of the slope stability of new or modified slopes, as these procedures are well-covered in a variety of textbooks and other comprehensive references and are well-established within the geotechnical engineering community. Instead, attention is focused on identifying the variety of slope stabilization methods commonly used, the level of subsurface information that should be collected, minimum factors of safety, and the level of monitoring to verify slope stability assessments for existing or modified slopes or progression of subsidence.

1.2 Problem Statement

Past practices for pipelines, as well as almost all other construction projects, have focused on avoidance of areas that have a reasonable probability of experiencing geohazards (defined as large ground displacements that may arise from slope failure, slope creep, earthquake triggered slope movement, and subsidence). This approach has been generally successful when there are limited restrictions on selecting a pipeline alignment. Deficiencies in the hazard avoidance approach can generally be traced to lack of knowledge at the time the alignment was selected, changes in environmental conditions (e.g., unusually high rainfall and loss of vegetative cover) or unexpected development near the pipeline alignment that leads to conditions that increase the potential for geohazards.

Avoiding potential geohazards is becoming increasingly difficult because of the inability to obtain landowner agreements, the lack of space in common utility corridors, environmental restrictions, incompatibility with existing land use, and/or public opposition. In route corridors where geohazards cannot be avoided, the potential risks associated with these hazards must be managed. Pipeline integrity management strategies to mitigate geohazards consist of: (1) design measures that improve the pipeline resistance to the geohazard, (2) measures that limit or control the severity of the geohazard, and (3) operational programs to monitor ground displacement or pipeline response and identify conditions that may warrant further engineering investigations or mitigation activities. Identifying the most appropriate mitigation strategy needs to be based upon specific hazard scenarios and operating circumstances.

Preparation of these guidelines included a thorough review of the current state-of-practice with respect to the above topics. Based upon this review, a list of central principles, presented in Table 1.1, was prepared as a basis upon which to frame specific recommendations within the guidelines.

1.3 OVERVIEW OF PROCESS

The general process for identifying actions to be considered in the design or assessment of a buried pipeline subject to ground displacement hazards from landslides and subsidence is illustrated by the flow charts in Figures 1.1 and 1.2 (and the accompanying notes in Tables 1.2 and 1.3). The guidelines treat landslide and subsidence hazards separately, primarily because of differences in causal mechanisms and available mitigation alternatives. Subsidence hazards are differentiated according to whether or not subsidence is a result of a natural process or the purposeful withdrawal or extraction of subsurface material, generically referred to as "mining."

1.3.1 Defining Performance Requirements

A fundamental question to be answered in performing an assessment of geohazards impacts for a new or existing pipeline is whether or not the desired level of performance has been achieved. The approaches in this document assume that the performance requirements for the pipeline have been defined in a manner that permits some quantitative goals for the occurrence frequency for pipeline damage and consequences. The most important consequence of the rupture of a natural gas transmission pipeline is the potential for release and subsequent ignition of gas in areas where there is a potential for injury or significant property damage. Other consequences include service interruption and the cost of repair and cleanup. Rupture of liquid hydrocarbon pipelines may have severe environmental consequences that are of equal or greater concern to the pipeline owner and govern the selection of performance requirements. However, users are cautioned that in the majority of cases, current capabilities for assessing risk from landslide and subsidence hazards is not compatible with rigorous quantitative risk assessment practices often applied in other areas of engineering.

This guideline does not establish acceptable levels of pipeline risk from which pipeline performance, expressed in terms of an acceptable annual probability of unacceptable response to geohazards, is defined. Decisions on performance requirements should be made on a case-by-case basis considering the governing risk measure (e.g., safety, economic loss, environmental damage, operational disruption) and local norms for risk tolerance. In most cases, it is considered reasonable to establish an upper-bound level of performance to be comparable to the performance requirements for non-pipeline projects, with similar consequences to the public (e.g., high-occupancy buildings, dams, bridges, LNG facilities).

Table 1.1 Basic principles used in developing geohazards guidelines

Issue	Principle
Means to Identify Hazard	Landslide hazard identification must necessarily begin with a subjective assessment of potential hazards along a pipeline route based primarily on observations and review of relevant information by experienced geological or geotechnical specialists. Means to identify locations of future geohazards rely upon identifying (a) areas that have failed recently, (b) areas that are actively moving, (c) areas that are very similar in terms of topography, geomorphology, hydrology, and soil properties to areas where evidence of past hazards is observed, (d) areas where changes in existing conditions (e.g. logging, deforestation, urban or industrial development) increase the risk of geohazards or (e) areas where activities leading to surface displacement may occur (e.g., mining, ground water withdrawal, oil and gas extraction).
Site-Specific Investigation	Additional landslide investigations should generally focus on confirmation of subjective assessments of landslide hazard and pipeline susceptibility to damage and may include activities to detect ongoing displacement, confirm the dimensions of the area undergoing displacement, and identify the depth of the critical slip surface.
Rate of Displacement	Unless specific criteria are met (to be defined during the course of the project), there is no basis to assume that slides that are exhibiting small displacement rates (e.g., less than a few cm/yr) can not exhibit episodic larger displacements rates (e.g., over 1 m in hours or days) over the typical life of a pipeline (50 to 100 years).
Landslide Hazard Definition	Reliable landslide hazard definition is restricted to identifying the location, dimensions, and depth of a potential slide; estimating potential damaging displacements that might occur in a rapid failure condition (one to several meters over the period of hours to days) is beyond the current state of practice.
Probabilistic Methods for Landslide Hazards	Probabilistic estimates of landslide hazard will typically be limited to “order of magnitude” annual likelihoods (e.g., ranges of 0.1%, 1%, or 10% per year) based largely upon subjective judgment of experienced professional geologists or engineers.
Subsidence Hazards Covered	Subsidence hazards that can practically be considered in pipeline design are related to subsurface withdrawal of material, drainage of saturated organic soils, and frost-heave and thaw settlement. Unlike the subsidence hazards noted above, subsidence related to sinkholes in karst terrains or hydrocompaction are often random and are generally difficult to identify and/or quantify. Approaches to address these hazards are site-specific and application specific and are not covered in detail in these guidelines.
Subsidence Hazard Definition	Available models for estimating surface displacements associated with removal of materials at depth are only sufficient for assessing pipeline response if they provide estimates of both vertical and horizontal displacement patterns.
Monitoring	Given the location of a potential landslide hazard that could be a threat to pipeline integrity, there are only two options: (1) take engineering steps to eliminate the hazard (grading, slope reinforcement, drainage) or (2) take operational steps to limit consequences of damage (periodic pipeline relocation, limit potential loss of contents). Either option requires post-construction monitoring although the type and frequency of monitoring are far less for option (1).
One vs. Multiple Hazards	There should be no difference in recommended actions between a pipeline project crossing one zone of ground movement and a project crossing multiple zones of movement with the same risk (likelihood of hazard x consequences).

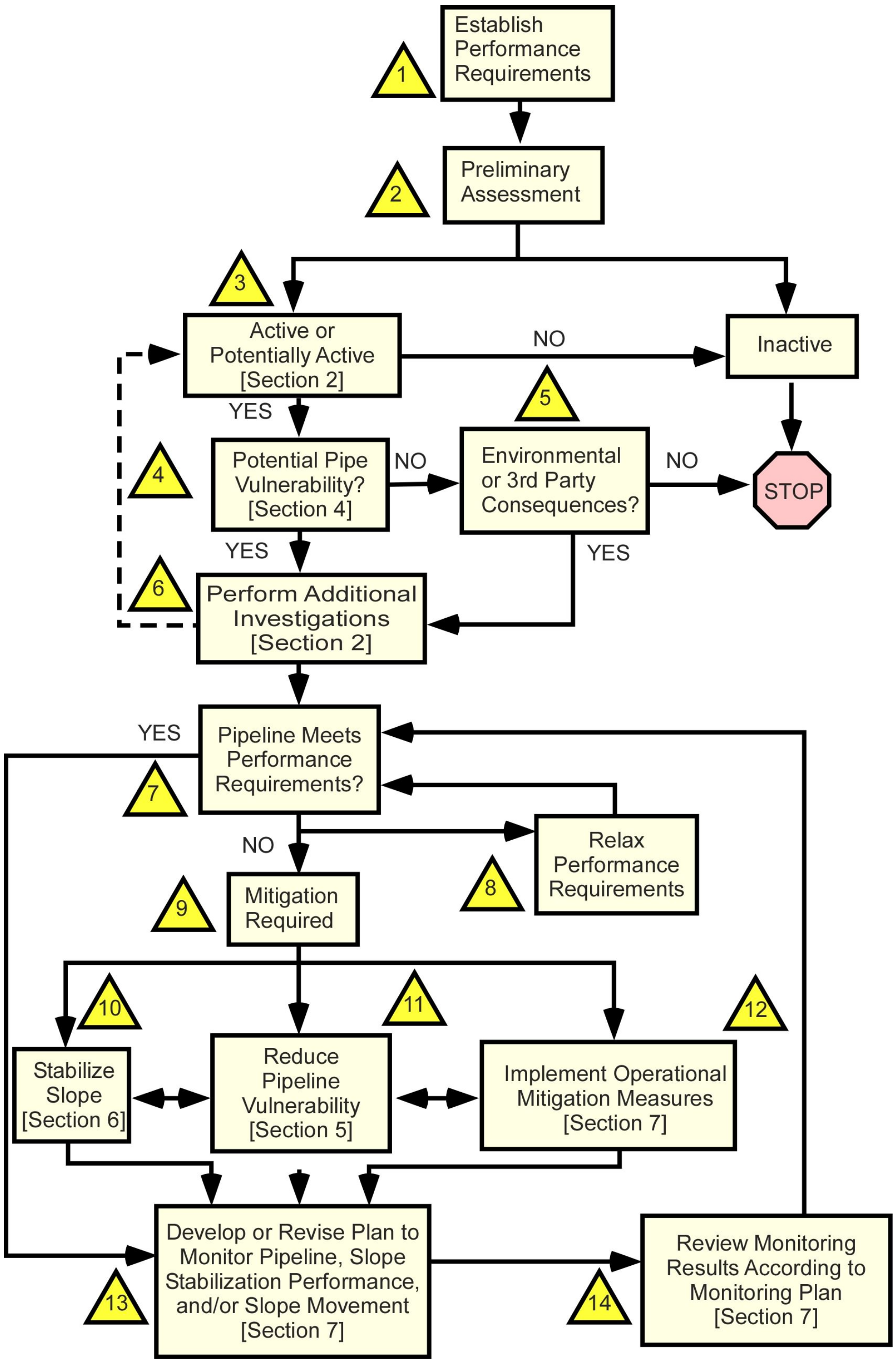


Figure 1.1 Flowchart of process for management of potential landslide hazards

Table 1.2 Notes for landslide assessment process

1	The level of performance required is expected to be stated in terms of an acceptable annual likelihood of exceeding a particular outcome (e.g., loss of pressure integrity, local pipe wall wrinkling) and may involve multiple requirements (e.g., annual probability of 10% for pipe wall wrinkling and 0.1% for loss of pressure integrity). These requirements will depend upon various pipeline-specific factors such as operational requirements, economic factors, environmental impacts, and impacts on adjacent communities and land owners.
2	Preliminary assessments will primarily be based upon desk top studies, making use of available imagery, mapping etc., supplemented with project specific route overviews. The goal is to identify landslide hazards or hazard areas along the route and conservatively define the hazard for preliminary pipeline vulnerability assessment (i.e. create a 'catalogue', or GIS database).
3	Examples of what constitutes active, potentially active, and inactive are provided in Table 1.4. Considering that landslides that are currently stable require some triggering event, which is likely to be highly uncertain, it is generally reasonable to consider landslides that are judged to have mean recurrence intervals of 10,000 years or greater (based upon and order-of-magnitude reference) to be inactive.
4	The purpose of these pipeline assessments is to determine whether or not the severity of the hazard can be dismissed on the basis that it is not severe enough to cause unacceptable performance for expected variations in pipeline properties and operating conditions, <u>assuming no limit on landslide displacement</u>.
5	Although the key concern is pipeline vulnerability to ground movement, there also needs to be an assessment of potential third party or environmental impacts from a landslide. The pipeline operator may not be responsible for the slide, but if it originates from or near their pipeline, they may be deemed to have contributed to it, or should have recognized the potential hazard, and have some liability for negative downslope effects.
6	Additional investigations will focus on improving estimates of landslide likelihood (e.g., through comparisons with landslide inventories, perhaps correlated with slope stability estimates) and hazard severity (e.g. slide dimensions, depth). The level of investigations will likely vary from site to site. Non-intrusive investigations may be suitable at some sites to provide the data necessary to finalize whether there is pipeline vulnerability or determine if performance requirements are met. Other sites may need full fledged intrusive investigations with the installation of substantial instrumentation with ongoing monitoring. This may require scheduling of data collection to be factored into overall project timelines. New data from these field investigations may change the assessment of slide activity and pipe vulnerability or may result in going directly to determining appropriate mitigation measures.
7	Pipeline vulnerability relative to performance requirements is reassessed, <u>assuming no limit on landslide displacement</u>, considering refined likelihood and severity of landslide hazard. If the pipe does not meet performance requirements based on unlimited displacement, then potential performance at various movement levels needs to be assessed. This is especially true if a re-route is a low feasibility option, and an extensive array of slope mitigation options cannot be counted on to 'fully remove' the hazard. Pipeline displacement capacity information is used to develop monitoring requirements and schedule the frequency of mitigation measures, such as relieving built-up strain in the pipeline.
8	In cases where desired performance requirements can not be met, some relaxation may be acceptable. For example, an initial goal of 0.1% annual chance of pipe damage might be relaxed to a goal of 1% annual chance of loss of pressure integrity.
9	Mitigation measures can include slope stabilization, changes to pipeline design, operational measures, or a combination of all three.
10	Slope stabilization will generally consist of combinations of a) increasing internal resistance through drainage, b) reducing driving forces through grading, or c) providing additional external resistance through buttressing, retaining walls, or tie-backs.
11	Changes to reduce pipe vulnerability can include increasing pipe wall thickness or material grade and reducing soil loading through pipeline coatings or specialized backfill specifications
12	Operational measures will generally consist of a) periodically alleviating pipe stresses through relocation or removal of backfill and/or b) providing means to minimize the consequences of pipe damage through containment and/or rapid shut-in of damaged pipe section.
13	Periodic monitoring is required for any mitigation method. The specific types of data to be collected and the monitoring frequency are highly dependent upon the type of mitigation measures and the relative potential for adverse landslide conditions. The development and periodic review of a detailed monitoring plan is an essential component of the overall mitigation process.
14	Monitoring data is fed into the overall monitoring plan. The data may result in changes to the monitoring frequency or operations parameters. The data needs to be used to continually assess if the pipeline still meets the performance requirements. If not, this may trigger when a strain relief is required, or further field investigations, or if other mitigation options need to be considered and implemented. Thus, a continuous loop is put in place of monitoring, updating, re-evaluation and change management.

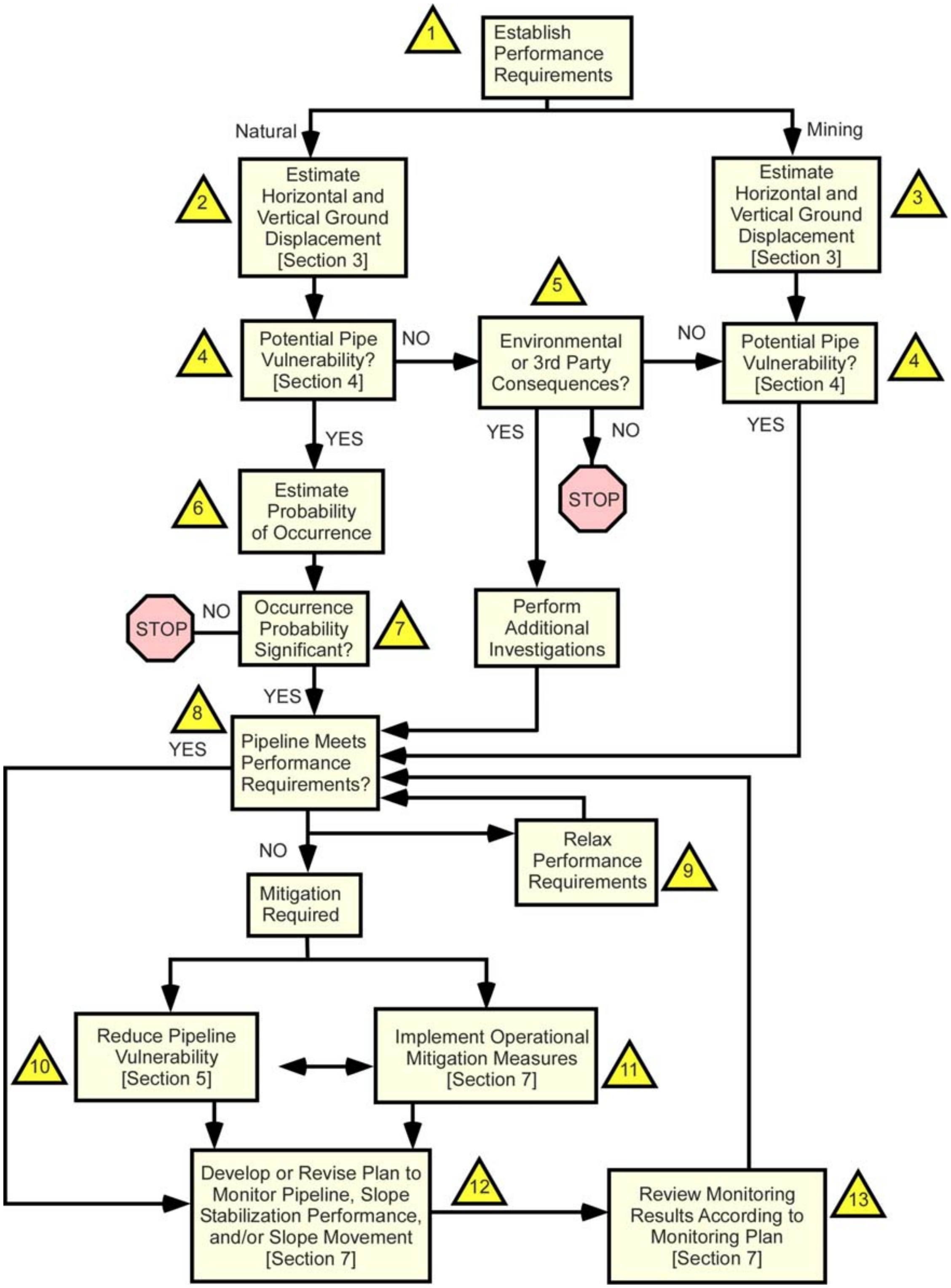


Figure 1.2 Flowchart of process for management of potential subsidence hazards

Table 1.3 Notes for subsidence assessment process

1	The level of performance required is expected to be stated in terms of an acceptable annual likelihood of exceeding a particular outcome (e.g., loss of pressure integrity, local pipe wall wrinkling) and may involve multiple requirements (e.g., annual probability of 10% for pipe wall wrinkling and 0.1% for loss of pressure integrity). These requirements will depend upon various pipeline-specific factors such as operational requirements, economic factors, environmental impacts, and impacts on adjacent communities and land owners.
2	Natural occurrences of subsidence (e.g., collapse in karst terrain) will generally be based upon observations of typical subsidence features in the region.
3	"Mining" is used as a general term to refer to any removal of solid or liquid subsurface material (e.g., water, salt, coal, oil). Mining subsidence will generally be based upon analytical models (typical for subsidence from coal mining) or historical measurements (typical for subsidence from ground water or oil withdrawal). All other forms of subsidence are considered "Natural".
4	Although the key concern is pipeline vulnerability to ground movement, there also needs to be an assessment of potential third party or environmental impacts from subsidence. The pipeline operator may not be responsible for actions that lead to subsidence, but if it originates from or near their pipeline, they may be deemed to have contributed to it, or should have recognized the potential hazard, and have some liability for negative effects.
5	Pipe vulnerability will typically be assessed using analytical models that explicitly account for non-linear soil-pipeline interaction.
6	The probability of occurrence for natural subsidence events will typically be based upon observations of the variation in event occurrence over some historical record.
7	The purpose of these pipeline assessments is to determine whether or not the severity of the hazard can be dismissed on the basis that it is not severe enough to cause unacceptable performance for expected variations in pipeline properties and operating conditions.
8	Pipeline vulnerability relative to performance requirements is reassessed considering refined likelihood and severity of the subsidence hazard. If the pipe does not meet performance requirements based on unlimited displacement, then potential performance at various movement levels needs to be assessed. This is especially true if a re-route is a low feasibility option, and an extensive array of slope mitigation options cannot be counted on to 'fully remove' the hazard. Pipeline displacement capacity information is used to develop monitoring requirements and schedule the frequency of mitigation measures, such as relieving built-up strain in the pipeline.
9	In cases where desired performance requirements can not be met, some relaxation may be acceptable. For example, an initial goal of 0.1% annual chance of pipe damage might be relaxed to a goal of 1% annual chance of loss of pressure integrity.
10	Pipe design changes can include increasing pipe wall thickness or material grade and reducing soil loading through pipeline coatings or specialized backfill specifications
11	Operational measures will generally consist of a) periodically alleviating pipe stresses through relocation or removal of backfill and/or b) providing means to minimize the consequences of pipe damage through containment and/or rapid shut-in of damaged pipe section.
12	Monitoring is required to identify the occurrence of natural subsidence events and confirm that subsidence related to mining is consistent with expectations.
13	Monitoring data is fed into the overall monitoring plan. The data may result in changes to the monitoring frequency or operations parameters. The data needs to be used to continually assess if the pipeline still meets the performance requirements. If not, this may trigger when a strain relief is required, or further field investigations, or if other mitigation options need to be considered and implemented. Thus, a continuous loop is put in place of monitoring, updating, re-evaluation and change management.

Defining performance requirements and the determining whether or not those requirements have been met is difficult to undertake in a strictly quantitative framework. There is considerable uncertainty inherent with available methods for estimating ground displacements. There is also considerable uncertainty in estimating pipeline response to ground displacement related to soil strength parameters, analytical methods, and pipeline performance under specific levels of strain induced by ground displacement. For these reasons, costs and benefits may be very difficult to assess.

In general, the goal for the design of natural gas transmission and liquid hydrocarbon pipelines is to achieve a design that is balanced with respect to safety and economic feasibility. The design should take into account the nature and importance of the project, cost to implement the desired design, and risk assessment centering around such items as public safety, loss of product or service, and damage to property and the environment. The desired level of performance for pipelines subjected to ground displacement hazards is typically determined on a project-by-project basis. Furthermore, the relative importance of the factors considered in establishing performance requirements will vary widely depending upon the local norms and regulatory requirements of the region in which a pipeline is located.

1.3.2 Overview of Landslide Hazards Assessment and Mitigation

The general process for assessing multiple landslide hazards along a pipeline route and identifying appropriate mitigation measures is illustrated in the flow chart of Figure 1.1 and the process diagram provided in Figure 1.3.

Landslide hazard identification is assumed to begin with a qualitative assessment of available mapping and aerial photography to identify and rank areas of existing or potential slope instability. This is the typical first step for pipeline projects in areas where there has not been previous development. Various methods used for this initial landslide hazard assessment are presented in section 2. Examples of landslide characteristics that can be useful in developing the qualitative estimate of the likely occurrence of a landslide are provided in Table 1.4. Landslides that are estimated to have mean recurrence intervals equivalent to the mean recurrence interval established by the pipeline performance requirements can be categorized as inactive. For example, a requirement that the annual probability of unacceptable pipeline performance be no greater than 0.0005 (1 in 2,000) would limit consideration of landslide hazards to those slopes with estimated mean recurrence intervals more frequent that 1 in 2,000 since this would accept a 100% chance of unacceptable pipeline performance. Considering that landslides that are currently stable require some triggering event, which is likely to be highly uncertain, and the limitation in establishing recurrence intervals for extremely rare slide movements, it is generally reasonable to consider landslides that are judged to have mean recurrence intervals 10,000 years or greater (based upon an order of magnitude estimate) to be inactive, regardless of the pipeline performance requirements.

This initial assessment should result in identifying the number of slides crossing or in close proximity to the pipeline alignment and a ranking of the slides according to the estimated probability of unacceptable pipeline performance. Such estimates may include a wide variety of factors but the most important are generally the annual likelihood of movement, the slide dimensions, the expected direction of slide movement relative to the pipeline alignment, an estimate of the likely impact of landslide movement on the pipeline, and the consequences of unacceptable pipeline performance. Initial assessments will typically be highly qualitative.

Table 1.4 Characteristics considered in assigning qualitative probability of landslide movement (adapted from Australian Geomechanics Society, 2000; Turner and Schuster, 1996; Lee and Jones, 2004; Nadim et al., 2005)

Characteristics	**Annual Probability**	**Qualitative Descriptors**			**Representative Average Slope Stability Factor of Safety (±0.1)**
Landslide is imminent; landslide form is well defined; material is continuously moving or has evidence of episodic movement within past 5 years; main scarp sharp and unvegetated; landslide expected during the life of the pipeline	greater than $2(10)^{-1}$ (1 in 5)	Certain	Active	Active, reactivated, or historic	Less than1
Landslide form is well defined: evidence of triggering of movement from events that occur, on average, every 10 years to 50 years; main scarp sharp and unvegetated; landslide expected during the life of the pipeline	greater than 10^{-1} (1 in 10)	Probable			1.0
Clear evidence of slope movement within the past 100 years; landslide expected during the life of the pipeline	greater than 10^{-2} (1 in 100)	Possible			1.2
Main scarp sharp and partly vegetated to smooth and vegetated: slopes have no evidence of previous instability; potential for movement based upon observations of similar slopes, historical records, or analysis; landslide not expected during the life of the pipeline	greater than 10^{-3} (1 in 1,000)	Unlikely	Potentially Active	Dormant-young to dormant-mature	1.3
Main scarp sharp and partly vegetated to smooth and vegetated: slopes have no evidence of previous instability; no evidence of movement based upon observations of similar slopes, historical records or demonstrated by analysis; extremely remote chance of landslide during the life of the pipeline	greater than 10^{-4} (1 in 10,000)	Remote			1.35
Main scarp vaguely discernable; slopes have no evidence of previous instability; no evidence of movement based upon observations of similar slopes, historical records or demonstrated by analysis; possibility of landslide during the life of the pipeline can be ignored	less than 10^{-4} (1 in 10,000)	Negligible	Inactive	Dormant-old	1.4 or greater

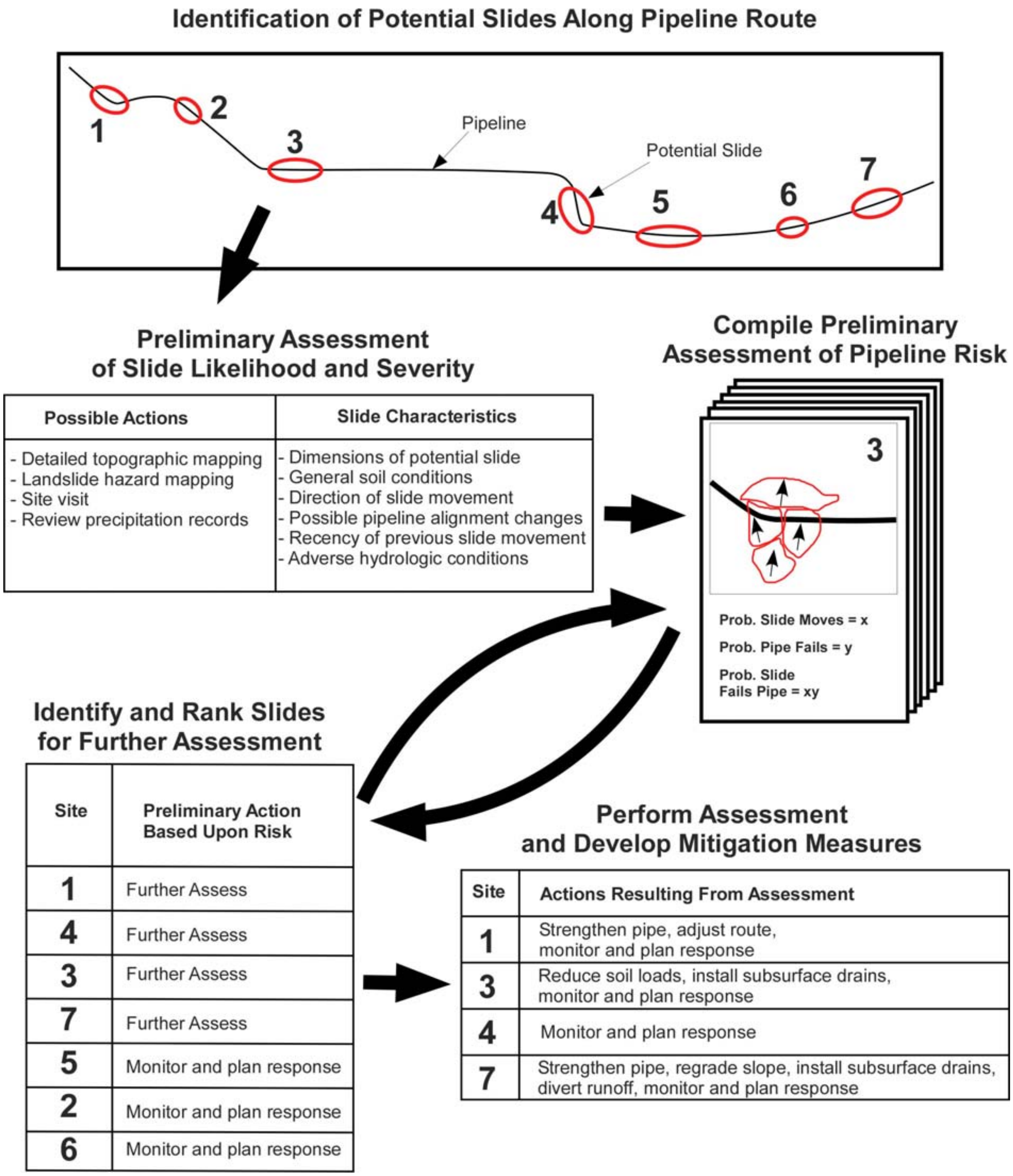


Figure 1.3 Process for addressing multiple landslide hazards along a pipeline route

It will often be beneficial to develop an approach that documents the reasoning behind the qualitative assessment. There are a variety of approaches that have been successfully applied to past pipeline projects. One approach, illustrated by a simple example in Figure 1.4, is to use an event tree to represent the factors and weighting contributing to the estimated likelihood of unacceptable pipeline performance. In the example in Figure 1.4, two event trees are provided to estimate the annual probability of unacceptable pipeline performance related to landslide movement triggered by heavy precipitation or erosion of the toe of the slope. Another commonly used technique relies upon quantitative scoring based upon various attributes of the landslide (e.g., age, steepness, depth, length). A full discussion of this topic is provided in Lee and Jones (2004).

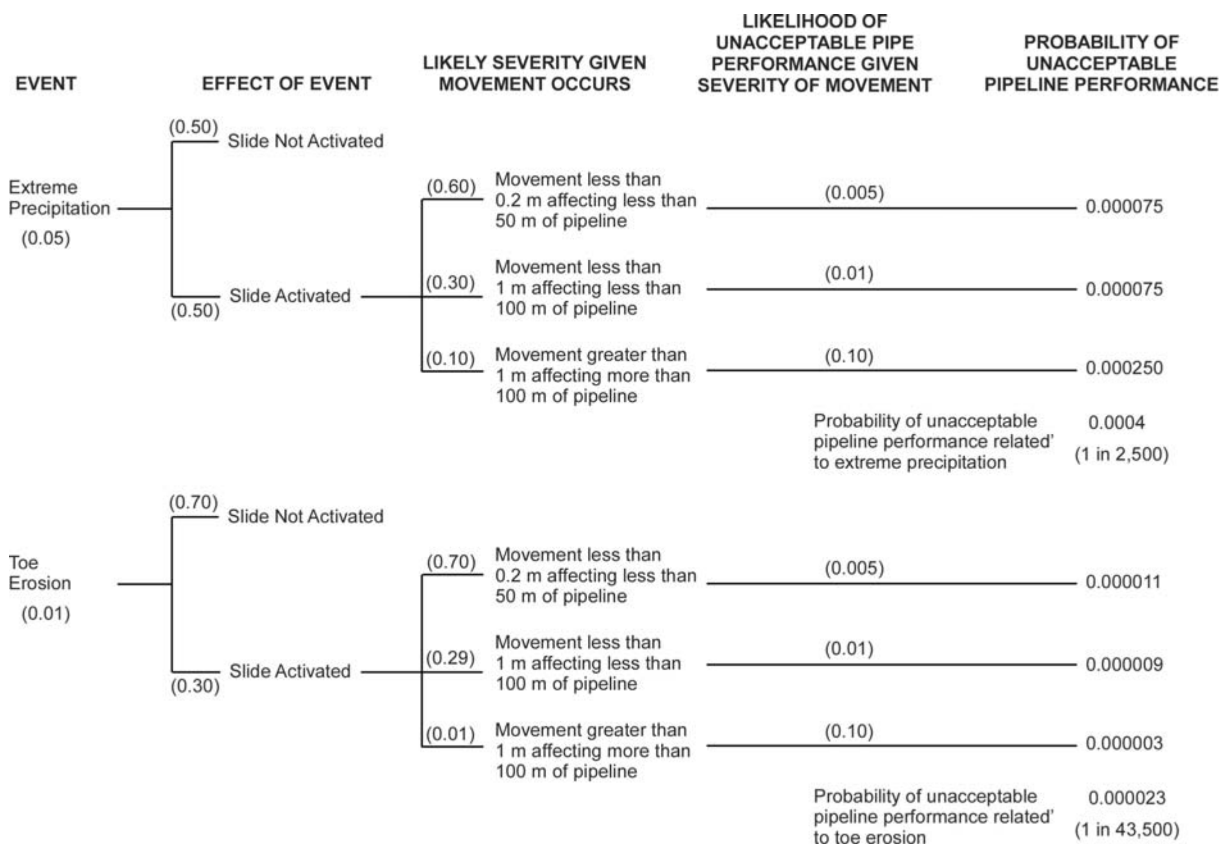


Figure 1.4 Example of the use of event trees to qualitatively assess probability of unacceptable pipeline performance

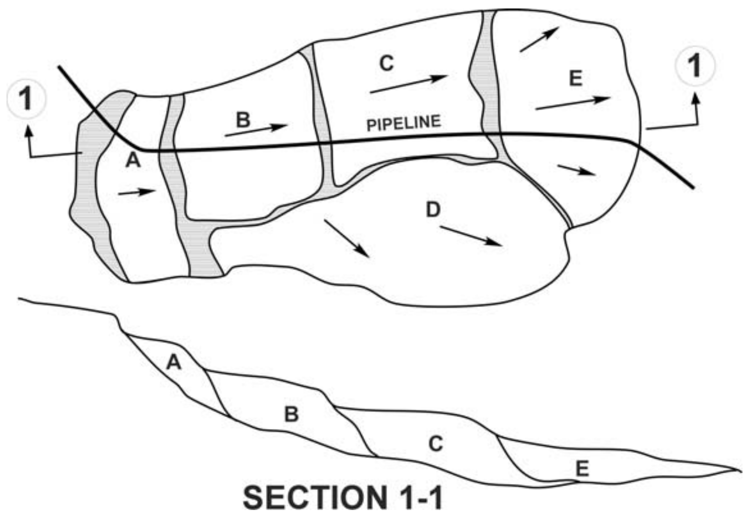
From the initial ranking, potential slide locations will be identified for which more detailed assessment of landslide hazard will be carried out (e.g., field investigations, slope stability calculations, landslide hazard mapping). The purpose of the additional investigations is to confirm whether or not a credible slide hazard exists, develop a better understanding of the key characteristics of credible landslide hazards, and conduct more detailed assessments of the pipeline response to the hazard. Information on the slide dimensions, the expected direction of slide movement relative to the pipeline alignment, and soil strength allow a preliminary assessment to be made of the level of vulnerability of a pipeline to slide movement. As illustrated in Figure 1.3, it is likely that the additional investigations will modify the risk ranking and change the number and priority of landside locations for which some mitigation will be necessary. In some cases, defining a set of possible landslide displacement scenarios for which pipeline response can be evaluated, as in the example in Figure 1.5, can be very useful in the ranking process. By evaluating pipeline response to various displacement scenarios, using the techniques described in section 4, a likelihood of pipeline failure can be estimated and used to assist in the relative risk ranking identified in Figure 1.3. It is important to note that specifying potential displacement scenarios, as illustrated in Figure 1.5, is for the express purpose of assisting in relative risk ranking. As noted in the discussion of the basic principles of this guideline, current practice is not capable of providing reliable estimates of landslide displacement.

Simplified hand calculation procedures can be used for a preliminary assessment if the pipeline alignment is straight through and beyond the zone of ground movement, the depth of soil cover and soil strength properties are constant, and the direction of ground movement is either purely parallel or perpendicular to the pipeline alignment. However, it is recommended that the assessment of pipeline response be performed using finite element analyses that explicitly account for non-linear soil-pipe interaction. Finite element analysis is required to assess more complex landslide scenarios such as illustrated in Figure 1.5. The recommended methodology for implementing such finite element analyses are presented in section 4.

Except for relatively rare circumstances, it is not possible to reliably estimate the amount of landslide displacement. Therefore, if pipeline design measures are investigated as the <u>only</u> means to reduce pipeline vulnerability, the pipeline design should demonstrate that the computed pipeline stresses or strains resulting from landslide displacement are acceptable for any magnitude of displacement. In general, implementing a pipeline design that can withstand unlimited displacement is either not possible or practical and pipeline design is not the sole mitigation measure. This does not mean that pipeline design is not a key component of an overall risk management strategy. Pipeline design measures that increase the displacement capacity directly affect decisions with respect to the selection of appropriate geotechnical mitigation measures, the frequency for monitoring ground displacement or pipeline response, and the need for other operational mitigation measures over the life of the pipeline.

If pipeline is assessed as not being vulnerable to the potential landslide hazard, an assessment of possible impacts of "third party" damage or environmental factors is necessary before the pipeline can be considered to meet the performance requirements. Third party damage generally refers to the consequences of actions undertaken by an individual or organization other than the pipeline operator or a contractor working on behalf of the pipeline operator. An example of third party damage relevant to the assessment of landslide risk would be excavations near the pipeline right-of-way that place additional driving forces on the slope, remove material providing stability to the slope, or alter the surface or subsurface hydrological characteristics of the slope. Examples of environmental factors that could alter the assessment of existing slope stability include possible changes in adjacent land uses, deforestation from fire or insect infestation, and atypical weather patterns related to climate change.

If it is determined that the pipeline can be constructed such that it is not exposed to potential landslide displacements of significance and there are no significant third party or environmental factors that would alter the assessment of landslide hazard, no further investigation is required and the pipeline can be considered to meet the performance requirements.



Scenario	A Displacement (m)	B Displacement (m)	C Displacement (m)	E Displacement (m)	30-yr Likelihood	Pipe Tension Strain	Pipe Compression Strain	Chance of Pipe Failure	Probability of Pipe Failure in 30 years
1	0.2	0.1	0.1	0.1	0.3	0.007	0.006	0.01	0.0030
2	0.2	0.3	0.4	0.6	0.2	0.010	0.012	0.05	0.0020
3	1	1	1	3	0.05	0.050	0.040	0.30	0.0150
4	0	0	0	0.3	0.15	0.005	0.007	0.04	0.0060
5	0.3	0	0	0	0.5	0.003	0.004	0.005	0.0025
6	0	0	0	0	0.25	0	0	0	0
				Sum of Scenario 30-yr Displacement Likelihood	1.0			Sum of Scenario 30-yr Pipe Failure Likelihood	0.0285

Figure 1.5 Example use of landslide scenarios to obtain an estimate of the probability of pipeline failure

Otherwise, there are two options: 1) obtain additional site-specific information to obtain an improved definition of the landslide hazard in the hopes that pipeline vulnerability will be eliminated or 2) implement hazard mitigation options. Additional observational and subsurface data will need to be collected for both options, although the type of data may vary as information to improve the pipeline vulnerability assessment is of lesser importance if a determination is made to mitigate the landslide hazard.

If the additional information provided confirms that the potential landslide displacements could result in unacceptable pipeline performance, there are three basic categories of hazard mitigation: 1) implement design changes to increase the pipeline resistance to ground displacement, 2) slope stabilization to reduce the likelihood of slope movement, or 3) implementation of operational measures to minimize the consequences of slope movement. These three general mitigation categories are treated separately in sections 5, 6, and 7, although in many cases, combinations of two or more categories will be implemented in order to meet the desired performance requirements.

As previously noted, improvements to the design of the pipeline are not sufficient to be considered a mitigation measure unless the pipeline can be designed such that the amount of landslide displacement is not important to the pipeline achieving the desired level of performance. If large landslide displacements could result in unacceptable pipeline performance, increasing the level of ground displacement that can be tolerated by a pipeline in a landslide hazard is only considered effective in increasing the time available to implement actions to mitigate the effects of landslide displacements on the pipeline. However, the additional time could be very important in determining the appropriate mitigation strategy. Increasing the displacement capacity of a pipeline by a factor of two or three could change the time to respond to a landslide event from several hours to several days to several weeks to several months.

Substantial uncertainty with respect to soil properties and potential hydrologic conditions will exist even after additional investigations have been completed. Therefore, some form of long term monitoring is recommended unless the hazard can be dismissed as a threat to the pipeline as discussed above to assure that the actual conditions are consistent with design assumptions (e.g., drainage is sufficient to control pore pressure, extent and direction of landslide movement has not changed, toe erosion or loading by nearby mass movement has not significantly increased the potential for slide movement).

1.3.3 Overview of Subsidence Hazards Assessment and Mitigation

While the process for addressing subsidence hazards in pipeline design generally follows the same framework as previously discussed for landslide hazards, there are some key differences related to the nature of subsidence hazards.

Subsidence arising from natural causes (e.g., hydrocompaction, sinkhole collapse) is largely a random hazard with the likelihood and severity primarily based upon observations of historical patterns. For natural subsidence hazards, it may be possible to demonstrate adequate pipeline performance based upon a statistical assessment of the probability of occurrence of a subsidence event of sufficient magnitude to lead to unacceptable pipeline response (e.g., sinkhole size larger than what can be spanned by the pipeline). In reality, situations suitable for statistical quantification of natural subsidence hazards are extremely rare. In general, the only practical approach to defining a natural subsidence hazard will rely on judgment and historical knowledge of past occurrences of subsidence.

Subsidence from mining differs from natural subsidence in two key respects: First, the location and time of occurrence of mining subsidence is largely known because of ongoing or planned mining activities. However, for subsidence related to planned long-wall coal mining activity, there will typically be some uncertainty regarding the direction at which mining will progress. Second, the amount of mining subsidence can be estimated based upon historical observations or analytical models.

Key parameters for defining natural and mining subsidence hazards include the length of pipeline exposed, the alignment of the pipeline through the subsidence zone, and the expected vertical and horizontal ground displacement relative to the pipeline alignment. A discussion of subsidence hazards and methods for estimating displacements are provided in section 3.

As with landslide hazards, pipeline vulnerability is assessed to determine whether or not the subsidence hazard poses a credible threat to pipeline integrity. Unlike the assessment of pipeline vulnerability for landslide hazards, the assessment of pipeline response to subsidence hazards is performed for a specific range of displacements.

Options to reduce pipeline vulnerability from subsidence hazards are largely limited to modifying the pipeline design or implementing operational measures to limit the likelihood of unacceptable pipeline performance (sections 5 and 7). While there are some examples of geotechnical mitigation measures, such as filling in voids from past mining activities or modifying potentially collapsible soils, the applications have been primarily focused on limited sites for construction of surface facilities and are rarely practical to implement along a pipeline alignment. Regular monitoring will typically be necessary to verify mining subsidence patterns are occurring as predicted or identify onset or full development of natural subsidence events.

2.0 LANDSLIDE HAZARD

As can be seen in Figure 2.1, substantial portions of the contiguous U.S. are susceptible to landslide hazards. This report provides a summary review of available methods for investigating individual landslides and landslide prone sites. Once regional methods have identified specific landslides or landslide prone areas that pose a major threat to facilities, site-specific methods outlined in this report can be used to further characterize the hazard. More detailed description and discussion of landslide hazard mapping, investigation and analyses are presented in Baum et al. (2008).

2.1 Regional Landslide Hazard Analysis

The common types of landslides for which location, size, and number per unit area or segment length along the pipeline alignment can be estimated using regional hazard analysis techniques are those whose failure depths are relatively shallow, for example average depths of no more than a few meters. These types of failures are falls and slides in rock and soil caused by earthquakes and severe precipitation events.

The accurate documentation of landslides is a key element of any analysis. A landslide inventory map at scales of 1:24,000 (1:12,000 scale is larger than 1:24,000 scale) or larger is necessary to accurately conduct a landslide-hazard analysis using slope, material properties (such as shear strength), or regional terrain attributes as input data. Such an inventory is compiled most effectively by the combination of field investigation and aerial photography at scales of 1:20,000 or larger (Figure 2.2). To permit assessment of the annual probability of landslide hazards, sets of stereo photographs covering a period of 25 to 50 years or more, if available, are desirable and often necessary.

Satellite imagery is increasingly available for many parts of the earth and at larger scales. However, most satellite imagery still has a resolution of 30 m or greater. The larger landslides could be mapped at the scales of 30-meter resolution, but many of the landslides triggered by an event (severe rainfall or earthquake shaking) would be too small to be detected at these scales. Even though some imagery is of sufficiently large scale to detect failures of one meter or less, satellite imagery is rarely in stereo. This makes it extremely difficult to interpret and map slope failures in their correct (i.e. within approximately 30 m) locations. In remote areas through which pipeline corridors are commonly located, landslide documentation is often sparse or nonexistent. Without suitable imagery to construct a representative and accurate landslide inventory, it is not possible to construct a landslide map and the quality of landslide hazard analyses may be impaired.

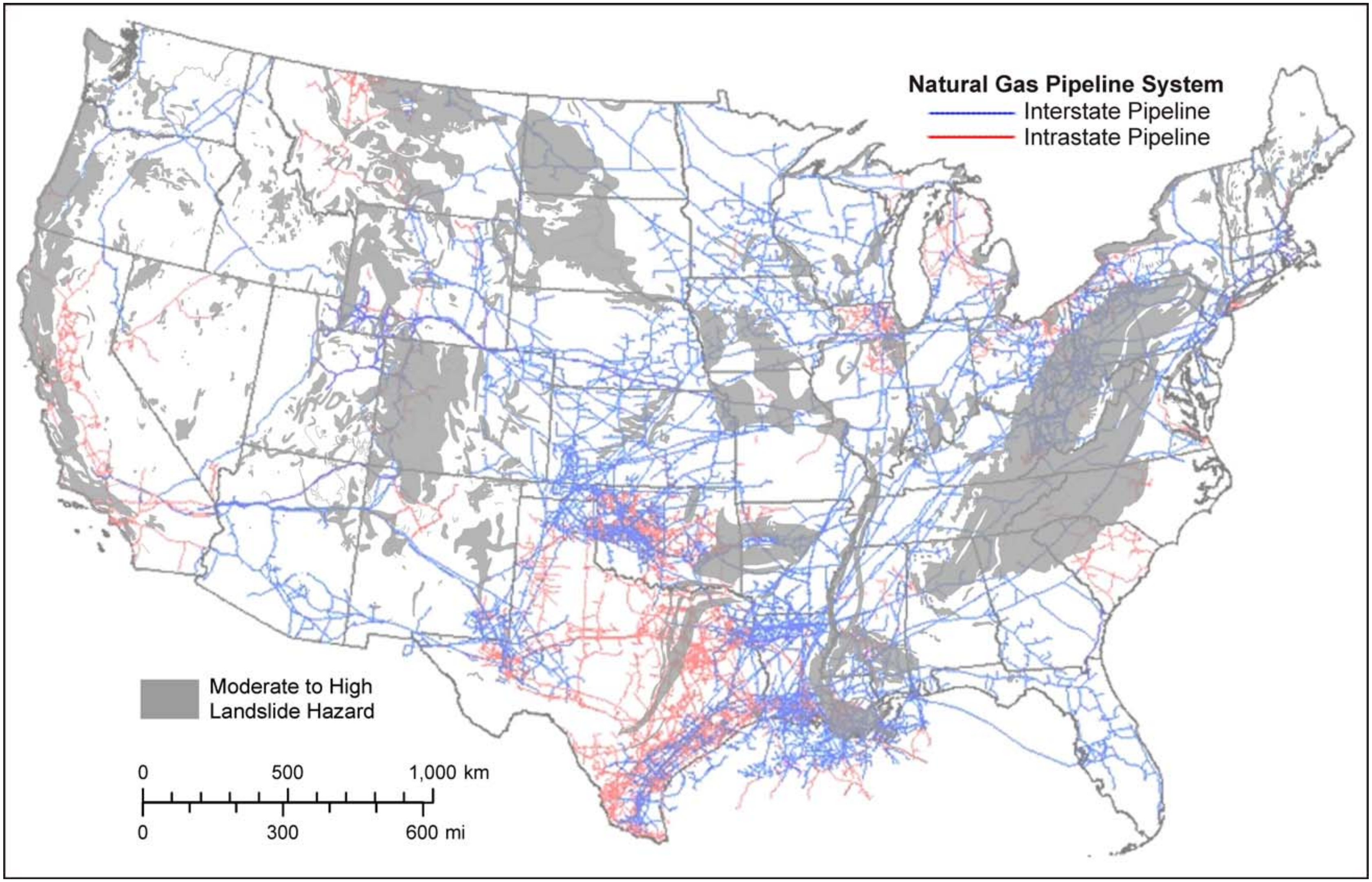


Figure 2.1 Co-location of landslide hazard areas and natural gas transmission pipelines in the contiguous U.S.

Methods to assess the susceptibility or hazard posed by landslides fall into two general categories: qualitative and quantitative. The qualitative methods can be separated into two sub categories: (1) those that evolve from field and aerial photographic investigations based largely on experience and judgment, and (2) those that are determined based upon comparisons of index factors or weighted parameters. Quantitative methods can also be separated into two general categories: (1) statistical methods and (2) methods that rely on deterministic or probabilistic geotechnical models to evaluate susceptibility or hazard. The above categories are highly generalized. A great degree of variation exists in both qualitative and quantitative methods. In many instances, qualitative and quantitative methods have been merged to introduce a significant degree of judgment and perception into numerical categorization.

2.1.1 Selection of Extent of Landslide Hazard Mapping and Analysis

Regardless of the mapping, investigation and analysis techniques used for determining the risk to pipelines due to landslides or other geohazards, an important question is how wide an area should be studied to develop a suitable and effective landslide risk assessment. At a minimum, a landslide risk assessment would be needed for the entire length of the pipeline transect or corridor. A more conservative approach would be to develop the history for all drainage basins that contribute directly to hazard along the pipeline. This approach results in a study area that has variable width along the pipeline, but it ensures that potential landslide sources are not overlooked by setting an arbitrary fixed width for the study area. Natural processes occurring within the drainage basin but hundreds of meters away from the actual pipeline can affect slope stability. The same can be said for human activities (excavation, grading, irrigation) occurring upslope or downslope of a pipeline.

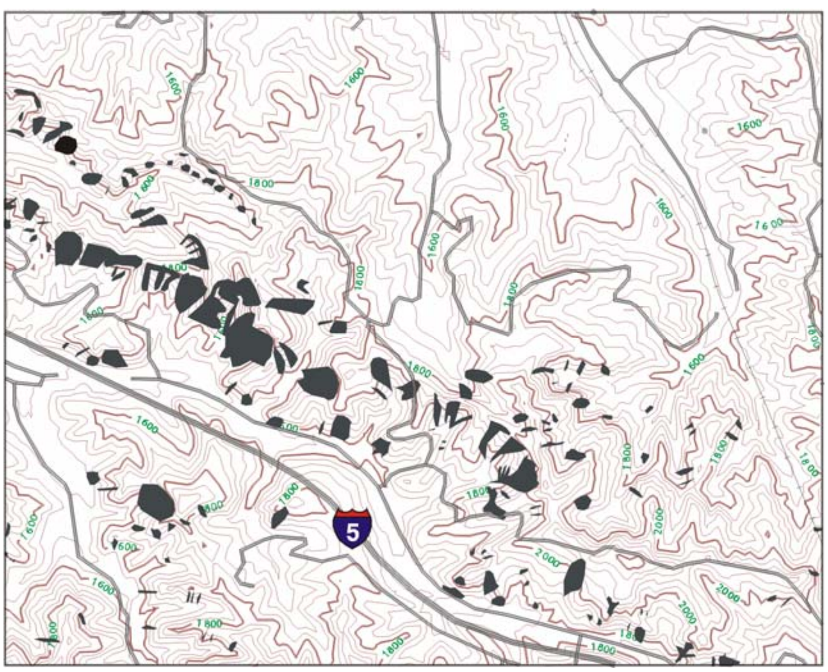


Figure 2.2 Landslide inventory from rock falls and rock slides triggered by the 1994 Northridge, California, earthquake (from Jibson et al., 2000)

2.1.2 Qualitative Methods of Analysis

Geomorphic Analysis

The first of the qualitative methods base landslide susceptibility or hazard on implicit determinations based on observation of landscape appearance made in the field or by use of aerial-based imagery. The assessment of susceptibility or hazard is derived from the evaluator's experience and recognition of morphological patterns that are similar to other unstable situations. Hazard criteria are generally implicit rather than explicit, and assessments produced by this process, although relatively rapid and including a large number of factors, are difficult to compare with those generated by other investigators (Figure 2.3).

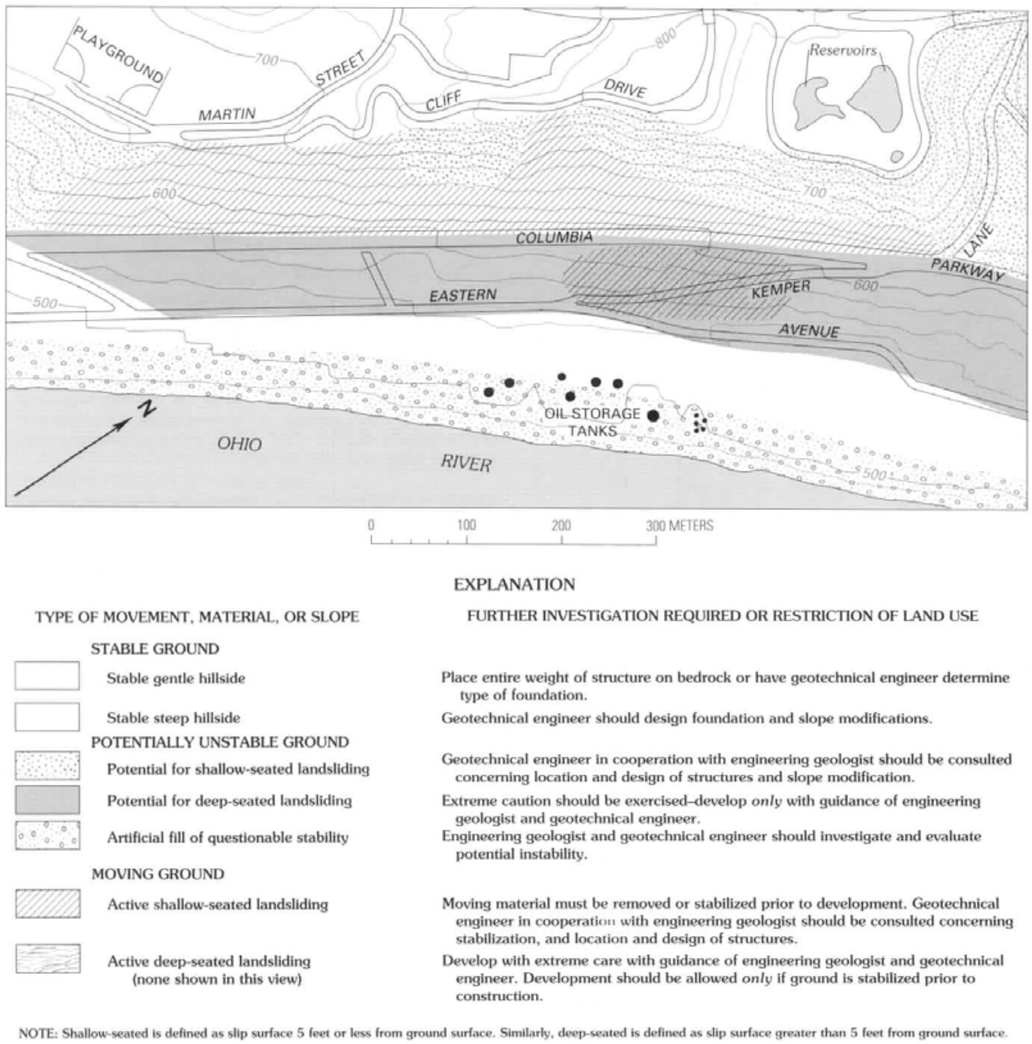


Figure 2.3 Relative stability map of part of Mt. Adams area in Cincinnati, Ohio (from Baum and Johnson, 1996)

Weighted Parameter Analysis

A second general type of qualitative analysis is that based upon a combination of weighted parameter maps. Parameters that influence the stability of slopes are mapped based upon professional judgment and assigned a weight in accordance with the relative contribution of the parameter to slope failure. The individual weighted parameter maps are combined to generate the final hazard map. This type of analysis has the advantage of specifying the parameters affecting slope instability and their relative contributions. It also allows automation of the process with the use of a GIS platform. However, it still retains subjectivity in establishing relative weights to assign to the various parameters.

An example of a weighted parameter criteria is that used by Harp and Noble (1993) to assign numerical scores to various fracture characteristics of rock slopes to assess rock-fall susceptibility and to use the criteria as a means to estimate hazard. Coe et al. (2005, 2007) used the rock mass quality criteria from Harp and Noble (1993) in combination with other weighted parameters to construct a hazard map for the Little Mill Campground near Provo, Utah (Figure 2.4).

Another such analysis by Coe et al. (2004) was used to evaluate the influence of terrain parameters (slope and elevation) in triggering landslides from Hurricane Mitch (October 1998) in Guatemala (Figure 2.5). After obtaining the numerical ratios (percentages of slope and elevation parameters with respect to landslide locations versus percentages of these parameters with respect to the total area) numbers from 1 to 5 were assigned to reflect the relative susceptibilities.

2.1.3 Quantitative Methods of Analysis

Statistical Analyses

The main limitation of the above methods is the subjective weighting of various parameters that influence the landslide process. The advantages of statistical techniques are that they allow the weights of the different mapped geological and topographical parameters to be determined by direct comparison with a landslide map. This process can be accomplished with a univariate or bivariate analysis where each parameter is compared separately to a landslide distribution or by multivariate analyses to numerically describe their respective influences on the landslide distribution or density (Figure 2.6). Correlation coefficients are then statistically evaluated to determine the degree of significance of each of the various factors considered in the analysis. However, statistical analysis methods often require considerable and detailed existing data on landslide events and causative factors, which may not be available for the area of concern. Further, because factors contributing to landslides can be interrelated, it may not be possible to determine the significance of the various factors with confidence.

Geotechnical Models

The deterministic analysis method refers to standard engineering slope-stability analyses carried out for specific sites. Physical properties of materials are quantified and serve as input in specific mathematical models, and factor-of-safety is calculated. The factor-of-safety is the ratio of the forces resisting slope movement to the forces driving it. Thus, factor-of-safety values greater than 1.0 indicate stability while those less than 1.0 indicate instability. Therefore, greater factor-of-safety values are taken to represent more stable slopes. These methods are discussed in greater detail in Baum et al. (2008). The deterministic analysis may utilize credible upper-bound and lower-bound ranges of selected input values to represent variability and uncertainty or represent conditions associated with a scenario event (e.g., heavy rainfall, reduced slope drainage).

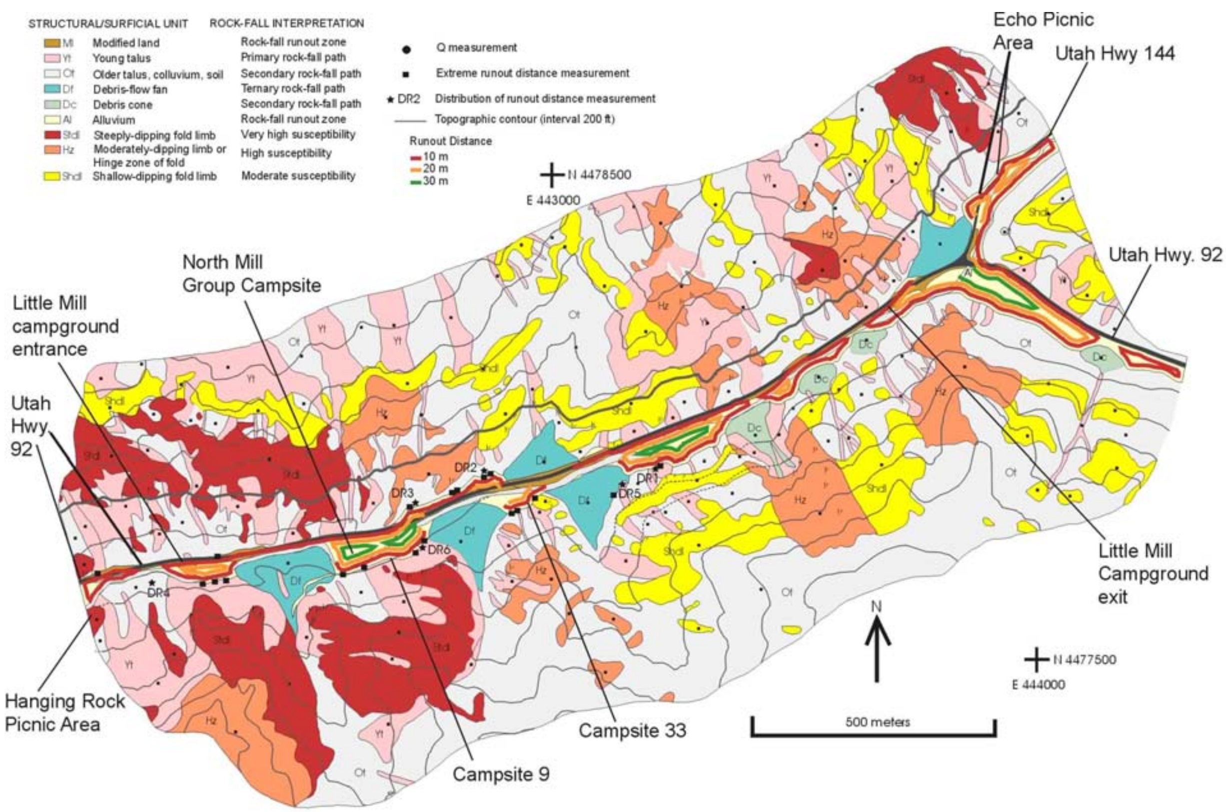


Figure 2.4 Rock-fall hazard map of Little Mill Campground, American Fork Canyon, Utah (from Coe, et al., 2005)

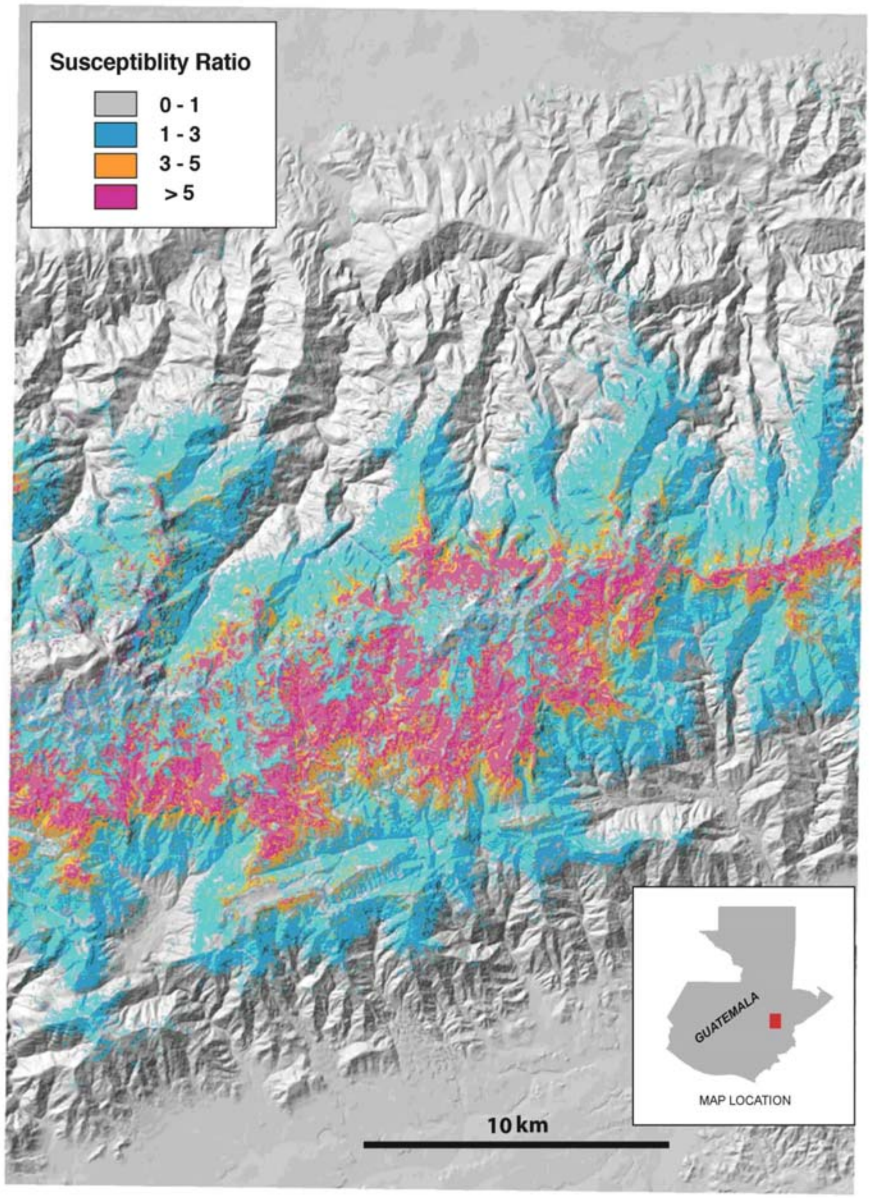


Figure 2.5 Landslide susceptibility map produced from ratio based upon elevation and slope (from Coe et al., 2004)

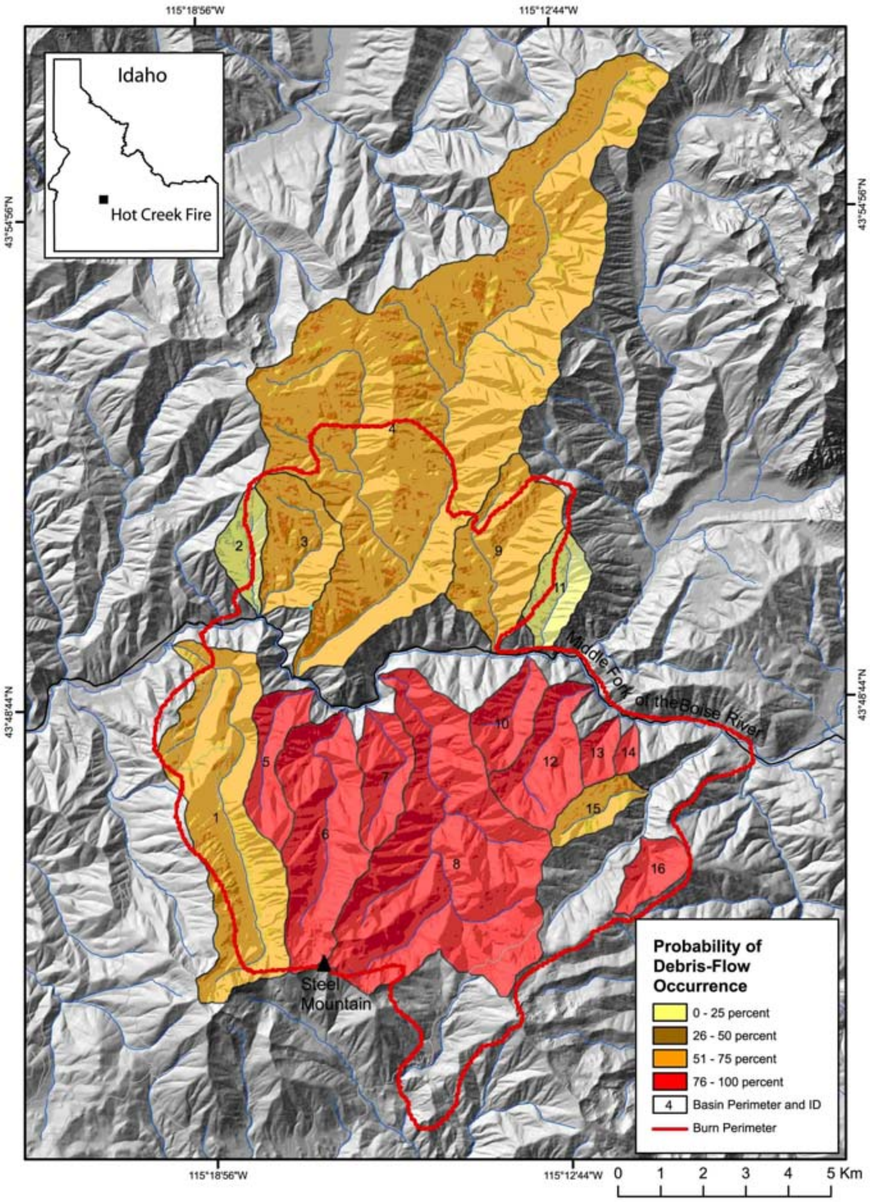


Figure 2.6 Map of probability of debris flow occurrence for basins burned by the Hot Creek Fire in response to a 1-hour, 10-year recurrence storm (from DeGraff et al., 2007)

Probabilistic geotechnical models refer to models that use standard geotechnical analyses that are coupled with a probabilistic evaluation, usually on a geographical information system (GIS) platform, to estimate factor-of-safety or a similar index of performance over a regional area. Most geotechnical models adapted to a GIS employ some form of probabilistic input based upon assumed or known variations in soil or rock strengths, water levels and other subsurface or topographic conditions in the calculation of factor-of-safety or in a comparison of factor-of-safety values with a distribution of known landslides to estimate probability of failure.

These methods are used for both shallow and deep-seated landslides. While there is no reason that regional models cannot conceptually describe deep-seated landslide hazard, there are several practical reasons why hazard analyses using geotechnical models of deep seated landslides are not nearly as reliable on a regional basis as geotechnical models used for shallow landslides. These reasons are discussed in the following sections.

2.1.4 Shallow Landslide Analyses

Numerous similar methods to estimate shallow landslide hazard exist that include the hazard posed by falls and slides in rock and soil and the shallow slumps and translational failures that form debris flows. All of these methods employ an infinite slope analysis which models slope segments as rigid friction blocks that are considered to be infinitely long in all directions.

There are a number of methods commonly used in GIS analyses to estimate the stability of slopes that are divided into grid cells. Many of these methods calculate the factor-of-safety of each cell. SINMAP (Pack et al., 1999) and SHALSTAB (Montgomery and Dietrich, 1994) are two similar programs that predict slope stability using an infinite-slope analysis. SINMAP uses ranges of rainfall and material properties expressed as uniform probability distributions.

Level I Stability Analysis (Hammond et al., 1992) is another infinite slope analysis developed by the US Forest Service that calculates a probability for failure of slope cells from different combinations of variables within the infinite-slope equation, each with their own probability distribution. This model uses essentially the same equation for factor-of-safety as SINMAP and SHALSTAB except that terms for tree surcharge (weight) and root strength are introduced.

Yet another method is Iverson's transient-response model (Iverson, 2000), which links a pore-pressure response function with the governing factor-of-safety equation. The pore-pressure response function is determined by applying a fixed rainfall intensity for a specified period of time into a one-dimensional infiltration equation using an estimate of soil hydraulic diffusivity. These models allow calculation of factor-of-safety at different depths in the soil column and at different times in the rainfall period, but require an estimate of the hydraulic properties of the existing soils (which can vary three to four orders of magnitude even within materials of uniform texture; Reid, 1997; Freeze and Cherry, 1979) and the initial pore-pressure distribution, parameters that are not commonly available for most slopes.

Harp et al. (2006) have used a simpler version of the infinite slope equation to construct a shallow landslide hazard map for the city of Seattle, Washington (Figure 2.7). The above model has also been used in areas of the world where little geotechnical test data are available.

The above methods of regional analysis have been applied to most common types of shallow landslides (falls, slides, and slumps in rock and soil). The shallow landslide types pose minimum hazard to pipelines that are buried unless their movement results in penetration or erosion to pipeline burial depths.

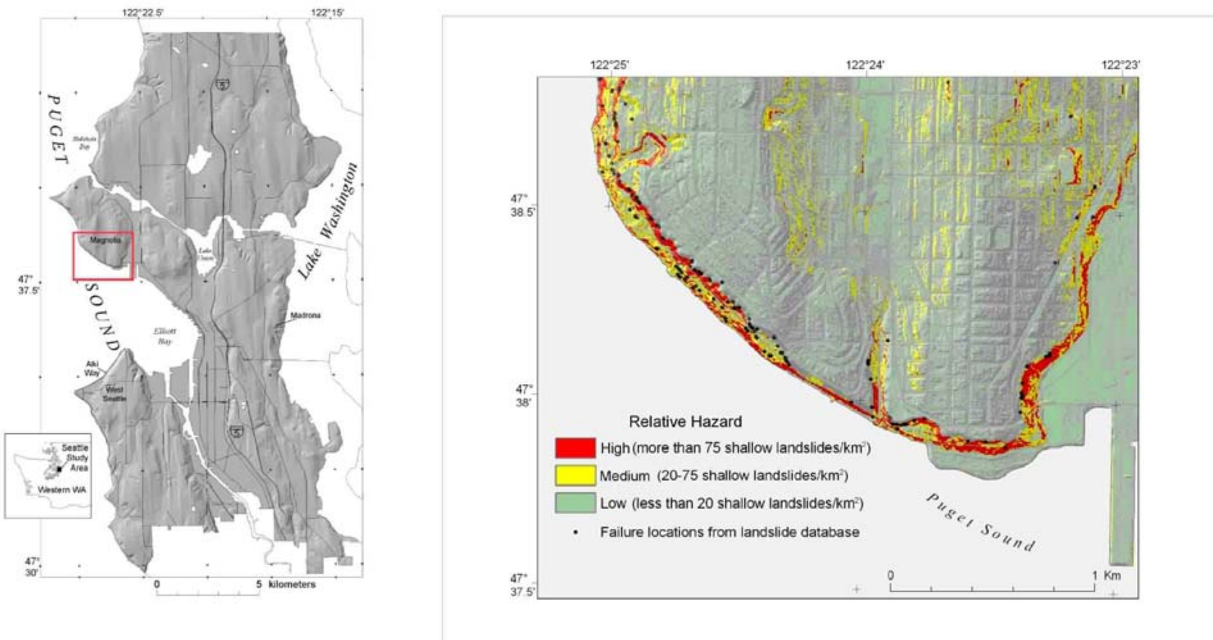


Figure 2.7 Portion of relative shallow landslide hazard map for Seattle (from Harp, et al., 2006)

2.1.5 Deep-Seated Landslide Analyses

Deep-seated landslides are the primary threat to pipelines because they typically involve rotational or translational movement in rock or soil large enough to threaten pipeline integrity.

As for the shallow landslides discussed above, both qualitative and quantitative methods of hazard analysis are used for deep-seated landslides. The field geomorphic, weighted parameters, and statistical methods described section 2.1.4 are applied to deep-seated landslide susceptibility or hazard in the same manner as shallow landslide susceptibility or hazard. Deterministic geotechnical analyses for deep-seated landslides on a regional scale are necessarily different than those used for shallow landslides. First, the use of the infinite slope analysis is not appropriate for deeper landslides. Secondly, there are more differences, structurally and geometrically, between one deep-seated landslide and another than between shallow landslide types. And finally, deep-seated landslides in a region tend to respond less to a single triggering event or group of events than shallow landslides which can occur over a wide area in response to a single triggering event (e.g., earthquake shaking). Deep landslide movements are more spread out over time than movements of shallow failures, making statistical analysis problematic.

Methods to evaluate the stability of specific deep-seated landslides are numerous and rely on adequate characterization of the slope, including sampling and testing of the material properties of the landslide mass and the slip surface, to formulate a reliable estimate of the stability of the landslide usually determined by some type of slope stability calculation.

Despite the tendency of deep-seated landslides to have much greater differences in geometries and mechanisms of failure than shallow landslides, methods to merge GIS analyses and slope-stability calculations for deep-seated landslides have begun to be employed on a regional basis. Miller (1995) and Miller and Sias (1997) have used conventional two-dimensional moment equilibrium analyses coupled with ground-water models to estimate the factor-of-safety throughout landslide terrain in watersheds in northwestern Washington. For these analyses, circular or elliptical slip surfaces along a regional grid parallel to slope were analyzed to determine the slip surface with minimum factor-of-safety for each grid point.

Other investigators such as El-Ramly et al. (2002) have employed similar two-dimensional conventional stability analyses coupled with statistical techniques to evaluate and minimize the variance of the various input parameters (shear strength, unit weight, pore pressure, etc.) so that probabilities of failure or "unsatisfactory performance" could be quantified. With these methods, results are highly dependent on the degree to which the input parameters can be specified and whether sufficient data exist to characterize the spatial variance of the input parameters.

Input for the various mathematical models in the above deterministic methods are acquired from actual measured data where possible and estimated where not. Most geotechnical data gathered even from a site specific slope-stability investigation would still be insufficient to use for application to evaluating slope stability over a much wider area. The most common techniques of estimating the distributions of properties such as shear-strength parameters, unit weight, and pore pressure assume a normal or lognormal distribution about an expected central value. In most cases, the ranges of values used to define the distributions are selected from what data exist and from judgment based upon experience.

2.1.6 Advantages and Disadvantages of the Various Susceptibility or Hazard Assessment Methods

Qualitative Methods

The main advantages of the methods of qualitative hazard assessment are that they are relatively rapid. The susceptibility or hazard is assigned based solely on the investigator's judgment and is either unspecified or is based upon a weighting of specified factors that affect slope stability. In either case, the assignment of weights or simply hazard itself is based upon judgment of the investigator and cannot be replicated by others. Although subjective, once the weights have been established, the process of overlapping weighted maps and developing a hazard map can be automated and performed on a GIS platform.

Statistical Quantitative Methods

Of the statistical methods of assessing landslide susceptibility and hazard, bivariate methods are the most straightforward. Simply comparing sets of mapped factors to a landslide map and determining the weight factor based upon the density of landslides captured by the separate factors is reproducible, especially if the weighting factors are directly proportional to the densities of landslides for the respective factors. The final overlay of factor maps to calculate the resulting hazard or susceptibility is easily accomplished within a GIS.

Multivariate methods of modeling landslide hazards gained popularity from their ability to assess the effect of numerous factors on the susceptibility of slopes to landslides either simultaneously or stepwise. All considered factors could be regressed against a mapped landslide distribution and correlation coefficients could be determined and susceptibility assigned based upon the individual correlations.

The advantage of the various statistical methods is that they are extremely systematic and reproducible once the different slope-related stability factors are defined and the data collected. However, the actual mapping of these factors and analysis of the data concerning the factors is often time-consuming and cumbersome (Aleotti and Chowdury, 1999). Carrera et al. (1991) remarked that the gathering of data and encoding of the various factors for a multivariate statistical study of a basin in Calabria (Carrera, 1983, 1989) required a great deal of time. They also stated that "black box" models such as their discriminant analysis of the Tescia Basin "do not unravel the internal structure of the process involved", because even with all of the variables included, the analysis is too simple. It also is basin specific and cannot be transferred to other basins with different geology and morphology (Carrera et al., 1991, p. 443). Use of these methods requires that there is a reliable landslide inventory (event-triggered or otherwise) for comparison with the various factors.

Deterministic/Probabilistic Quantitative Methods

The main advantages of these types of analyses are that uncertainties in the variables that affect slope stability can be taken into consideration. These analyses can incorporate modifications of slope geometry or other changes that affect the stability of the slopes due to construction activities that might occur in the development of a pipeline corridor. If sets of comprehensive data exist for variables such as shear strength, material unit weights, and levels of pore pressure, then a distribution of performance factors such as factor-of-safety can be reliably estimated. If not, which is usually the case, values of the means and variances of these variables are estimated as previously discussed in the section on shallow landslide analyses. Variables such as shear strength are often not normally or lognormally distributed, especially when considering formations which have interbedded layers of differing properties. When normal distributions are assumed for variables that may not have normal distributions, the results may have no more basis than the hazard map based solely on the implicit judgment and experience of the investigator, and give a false sense of quantitative assessment. This is especially true for the analysis of slopes with deep-seated landslides. Not only are material properties poorly known for most of these cases, but the failure geometries are also poorly known and are highly variable from one to another and are difficult, if not impossible, to generalize. Therefore, landslide hazard analyses for deep-seated landslides are inherently fraught with high (and often unknown) degrees of uncertainty. A summary of advantages and disadvantages of the various methods of landslide hazard analysis is presented by Aleotti and Chowdury (1999; presented in Table 2.1).

2.2 Engineering Geologic Mapping and Related Field Studies

The purpose of engineering geologic mapping in landslide investigations is to determine the dimensions and to identify and locate boundaries and other surface features (Figure 2.8) and geologic materials of the landslide. Mapping and related field studies also help to unravel the geological history of landslide development, which may result in estimates of magnitude and frequency of past movements. Engineering geologic mapping at various scales serves different purposes. Large-scale (1:500 to 1:1000) mapping shows the geologic (lithology, structure, geomorphology) and hydrologic (springs, sag ponds) details needed for study of individual landslides and landslide prone sites. Mapping at small (1:25,000 to 1:100,000) and intermediate scales show landslides and landslide prone areas in context of the regional and local geology and terrain.

Table 2.1 Advantages and disadvantages of different methods of landslide hazard assessment (after Aleotti and Chowdury, 1999).

Methods	Advantage	Disadvantage	Role of GIS
Field geomorphic analyses	Allow a rapid assessment taking into account a large number of factors.	Totally subjective, methodology uses implicit rules that hinder the critical analysis of the results.	Only as a drawing tool
Combination of index maps	Solves the problem of hidden rules. Total automation of steps. Standardization of data management	Subjective in attributing weighted values to each parameter.	Overlay of different maps
Statistical analyses (bivariate, multivariate, etc.)	Objective in methodology. Total automation of steps. Standardization of data management.	Systematic collection and analysis of data regarding different factors is cumbersome.	Analysis and map overlay
Probabilistic approaches	Allows consideration of different uncertainties. Quantitative in scope. Objective in scope and methodology. Provides insight not possible in deterministic methods.	Requires comprehensive data. Otherwise subjective probabilities required. Probability distributions difficult, especially for low level of hazard.	Analysis and map overlay

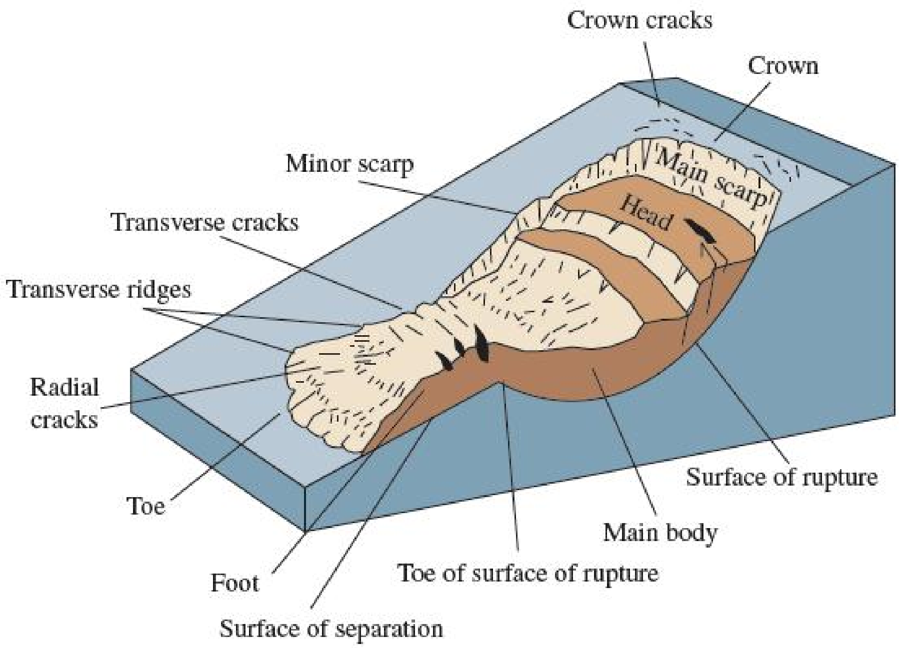


Figure 2.8 Block diagram showing the main parts of a landslide (after Varnes, 1978; Highland, 2004)

2.2.1 Small-Scale Mapping

Small-scale regional mapping was discussed in section 2.1, but some of the data collected in connection with regional mapping can contribute to an understanding of specific landslides. Although showing existing landslides on a map does not necessarily represent exactly where future landslides may occur, such mapping does help bound the ranges of several landslide characteristics for a particular area. For example, ranges in size and travel distance of landslides in a pipeline corridor can be determined from small to medium scale mapping (landslide inventory maps, Figure 2.9). The observed size ranges provide some constraints or guidance for estimating the potential widths of future landslides.

Detailed geologic maps showing a history of different periods of previous and recent landslides, sometimes referred to as multitemporal maps because they show landslides from multiple events spread over a period of years or decades, have been prepared within several regional areas of California. Similar maps exist for other areas, but coverage tends to be spotty. Information needed to ascertain landslide hazard can be obtained from analysis of a multitemporal landslide inventory map that portrays the distribution, type, and pattern of landslides and their changes in time.

Figure 2.9 Landslides from a major storm event near Seattle, Washington, red polygons represent shallow debris flows and earth slides, pink polygon represents a large deep-seated landslide (Baum et al., 2000)

The multitemporal map can be compiled from landslide inventory maps prepared through the analysis of stereoscopic aerial photographs of different ages and by use of field surveys (Reichenbach et al., 2004). Such aerial photographs are often not available prior to about the 1960's. Relatively high quality, small scale (1:10,000 to 1:25,000) stereo air photographs taken at 5 to 10 year periods or more frequently can be utilized to obtain information on the extent, progression and annual probability of occurrence of landslides along or adjacent to pipeline corridors. An estimate of the annual probability of landslide occurrence, even if approximate, can provide valuable input in the selection of alternative pipeline alignments, as well as decision making with respect to acceptance of existing risks or the need for and extent of mitigative measures. Depiction of prehistoric, historical, and recent landslides on detailed maps can depict the regional potential future landslide hazard (Wieczorek et al., 1999).

Recent applications of Light Detection and Ranging (lidar) to landslide mapping are also very useful. The lidar technique is based upon airborne scanning with a laser rangefinder and GPS ground control to produce high-resolution topographic data. Using algorithms for virtual deforestation (Haugerud et al., 2007) lidar data acquired during the leaf-off season is capable of producing detailed bare-earth digital elevation models. The quality of lidar mapping has steadily improved over the last several years as point densities of lidar surveys have steadily increased and postprocessing has become more sophisticated. The lidar topographic data are becoming available for more and more areas of the U.S. as public and private entities commission increasing numbers of lidar surveys. In some areas, these data are in the public domain and freely available; in areas where lidar has been acquired with private funding, the data may be available for purchase from a vendor.

Use of lidar-derived topography is particularly effective where pipeline alignments and adjacent landforms, including landslide features, are masked by extensive tree or vegetation cover or where excessively steep and/or otherwise inaccessible or dangerous terrain limits or precludes effective ground based mapping. Figure 2.10 is an illustration of a bare-earth map produced from lidar for a proposed pipeline alignment in Alberta. A natural photograph of the same area in Figure 2.10 is shown in Figure 2.11. This comparison shows the advantage of lidar in unmasking landslide features. The toe bulges and headscarps normally covered by vegetation are readily apparent in the lidar image. Even stereoscopic aerial photographs, depending on available scales, may not pick up the topography as well as lidar.

The lidar topography has been utilized extensively along the Vancouver to Whistler road and utility corridor to map steep rock bluffs and slopes. Elsewhere within coastal British Columbia, it has been used to establish accurate topography and assess potential slide features within heavily tree covered slopes (Butler, written comunication, 2007). Schulz (2004, 2005) used lidar-derived imagery to map landforms in Seattle, Washington, that were created primarily by landslides. These landforms include landslide deposits, head scarps, and denuded slopes that were created by prehistoric landslides that have occurred since the retreat of the last glacier. The spatial densities of reported historical landslides within the lidar-mapped landforms provide the relative susceptibilities of the landforms (particularly landslide head scarps and deposits) to landslide activity in the recent past. The spatial densities also provide reasonable estimates of future landslide susceptibility.

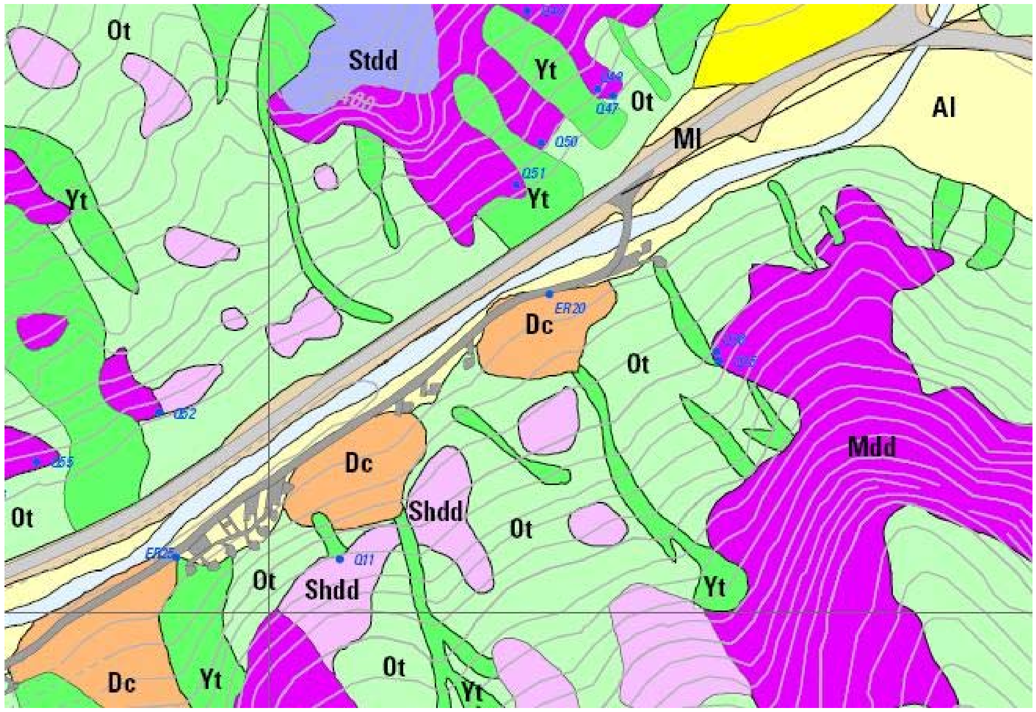
Although buried pipelines are not usually subject to damage by rock fall, aboveground facilities such as pumping stations and valves may be exposed to rock falls. Various techniques are available for engineering geologic mapping and related field studies to characterize the potential for rock fall hazards (e.g., Coe et al., 2005). Three dimensional analysis of rock slope stability has been developed using joint directions, slope orientations, and friction angles with a stereo net to create Markland plots (Markland, 1972).

2.2.2 Large-Scale Mapping

Detailed observation of bedrock, field developed cross sections, classes of slope stability, and surface water features are useful for geologic mapping of landslides and landslide prone areas (Keaton and DeGraff, 1996). Details to be included in a large-scale engineering geologic map of a landslide depend somewhat on the landslide types and processes involved. Several landslide classification schemes are used worldwide; one of the most widely used is the Varnes classification, which is based upon material and process (Varnes, 1978; Cruden and Varnes, 1996). The different types have different three-dimensional forms, but the types that most commonly damage pipelines have features similar to those depicted in Figure 2.8.

Figure 2.10 A lidar image of proposed pipeline alignment (courtesy of TransCanada Pipelines Limited)

Figure 2.11 Natural image of same slope as in Figure 2.10 (courtesy of TransCanada Pipelines Limited)



LEGEND	
Al	alluvium mixed with rockfall debris
Ot	older talus, colluvium, soil
Yt	young talus
Stdd	steeply dipping limestone
Mdd	moderately dipping limestone and/or fold axial zone
Shdd	shallow dipping limestone
Dc	debris cons
Df	debris fan
Ml	modified land
Q50	measurement of rock mass quality
ER20	measurement of extreme rockfall runout

Figure 2.12 Engineering geology of a rockfall prone area (Coe et al., 2005, plate 1)

Complex landslides can be depicted on detailed scale maps (Bogaard et al., 2000; Chelli et al., 2005). Major structures and features of the landslide emerge from mapping individual cracks, scarps, lateral shear zones, and so on. Areas of active or potential enlargement are identified by mapping small fractures outside the main body of the active landslide.

Similar levels of detail can be portrayed in engineering geologic mapping of a potential landslide area. Although landslide features may not be present, detailed mapping can be used to show geologic structures (faults, folds, joints), variations in lithology, zones of weathering or alteration, depth to bedrock, seepage zones, and other features relevant to slope stability, most notably any evidence of recent or ongoing ground deformation. Locations of boreholes, trenches, test pits, measurements, geophysical surveys, and instruments can also be shown on large-scale engineering geologic maps of landslide and potential landslide areas.

2.3 Landslide History

Information on landslide history or recurrence of movement constitutes the basis for most estimates of temporal landslide probability. Absolute or relative ages of landslides (or rather, the age of their last movement) are also used to make a preliminary assessment of their stability. Landslides that have not moved in hundreds or thousands of years are commonly assumed to be more stable than those that have moved more recently. However this approach must be used with caution because climate extremes, increased erosion rates, and human activities such as irrigation, grading, excavation or other changes to the land surface can invalidate this assumption. A complicating factor for some landslides is recent activity in only one part of the mass with geomorphic features over the rest of the mass that suggest that part has been dormant for hundreds to thousands of years.

Several techniques are available for partially reconstructing the history of movement in landslide areas. These include use of crosscutting relationships and degradation of features (scarp, hummock, etc.) to define relative ages of deposits (McCalpin, 1984), as well as methods for obtaining "absolute" ages. Approximate ages of past movement can be provided by radiometric ages of buried soils (Madole, 1996), datable materials (wood, bone, or charcoal) embedded in landslide deposits (Chleborad, 1996) and organic rich deposits that have accumulated in sag ponds and depressions that have formed on the surface of a landslide (Alexandrowicz and Alexandrowicz, 1999), or in the lacustrine sediments that have accumulated upstream of a landslide that has dammed a valley (Schuster and Pringle, 2002). Dendrochronology (Stoffel, 2006), lichenometry (Bull et al., 1994), pollen analyses (Adam, 1975; Baron et al., 2004), and similar techniques have also been used for estimating ages of landslides. Dendrochronology is capable of giving more precise ages than other methods, but corrections are needed when determining ages of young surfaces (Pierson, 2007), and certain types of trees are unsuitable for dendrochronology (e.g., scrub oak). It should be kept in mind that ages determined by any of these methods are approximate and subject to various limitations. Selection of sites for collecting datable materials requires a clear understanding of the morphology and internal structure of a particular landslide, as well as an understanding of which locations will give minimum ages and an awareness of other potential difficulties (Van Den Eeckhaut et al., 2007). The laboratory cost of obtaining radiocarbon ages is about $300-$600 (US dollars, 2007) per sample, depending on sample size and pretreatment requirements. In areas where historical data on landslide occurrence is unavailable, the value of a landslide history constrained by radiocarbon or other ages may far exceed the cost of obtaining samples and performing the laboratory testing.

Historical records are also useful for identifying major episodes of past movement, particularly in areas that have been occupied by humans for long periods of time. Unfortunately long historical records of landslide activity are relatively rare.

2.4 Landslide Movement Rates

Rates of landslide movement range from imperceptibly slow (millimeters per year) to extremely rapid (many meters per second). Based upon previous work by Varnes (1978), Cruden and Varnes (1996) proposed a landslide velocity scale (see Figure 2.13). Rapidly moving landslides (velocity classes 5, 6, and 7 in Figure 2.13) include rock and debris avalanches, debris flows, rapid earth flows in sensitive clays, rock falls, and some rockslides. Although small rock falls and shallow debris flows are unlikely to damage buried pipelines, avoiding large, rapidly moving landslides and their effects is critical in preventing damage to pipelines. Nearly all types of landslides may display long-lasting slow movements. However, slow landslides (velocity classes 1, 2, and 3) most commonly include translational landslides in stiff clays and other fine-grained deposits (earth slides and earth flows) as well as many deep-seated landslides and spreads, complex movements in rock masses, and some slides in granular soils (Picarelli and Russo, 2004).

Velocity Class	Description	Velocity (mm/s)	Typical Velocity
7	Extremely Rapid		
		5×10^{3}	5 m/s
6	Very Rapid		
		5×10^{1}	3 m/min
5	Rapid		
		5×10^{-1}	1.8 m/hr
4	Moderate		
		5×10^{-3}	13 m/month
3	Slow		
		5×10^{-5}	1.6 m/year
2	Very Slow		
		5×10^{-7}	16 mm/year
1	Extremely Slow		

Figure 2.13   Landslide velocity scale (after Cruden and Varnes, 1996)

Slow to extremely slow movements deserve further attention because of their potential to damage pipelines over time as cumulative displacement gradually increases, or as off right-of-way landslides move closer to the right-of-way and the pipeline. Even though the movement of many slow landslides appears to be relatively steady, detailed monitoring has shown that movement may be episodic or that movement rates may vary greatly over timescales ranging from hours to years (Keefer and Johnson, 1983; Kalaugher et al., 2000; Coe et al., 2003; Petley, 2004; Picarelli and Russo, 2004). These changes in rate of movement result from external factors such as precipitation and erosion as well as internal changes in the landslide mass that result from deformation.

In new landslides (first time failures), a period of slow but gradually accelerating movements typically precedes failure. Once failure occurs movement accelerates rapidly, and the newly released landslide mass moves abruptly (Picarelli and Russo, 2004). Subsequent movements tend to be slow, but periods of relatively rapid movement, known as surges, have been observed in many slow landslides. Pore pressure increases and changes in external loading acting together or separately have been identified as causing surges (Keefer and Johnson, 1983; Kalaugher et al., 2000). The surges may last several hours or days, and commonly result in displacements of several decimeters to several meters. Major shear distortions have been observed during the early stages of displacement surges (Kalaugher et al., 2000).

The form and nature of the deposits provide a general indication of rate of movement of past landslides. In other words, it is often (but not always) possible to distinguish deposits of debris flows, debris avalanches, and other rapidly moving landslides from deposits of slow landslides or other processes. Debris flows usually have lateral levies and the main deposits have characteristic fan shaped morphology; the deposits typically have large clasts supported by fine-grained matrix (Cruden and Varnes, 1996; Pierson, 2005). Slope and materials also give some indication of potential rate of movement. For example, most landslides in high plasticity clay on slopes flatter than 12°-15° usually move at slow to moderate rates (velocity classes 1, 2, 3, and 4); however as noted previously, even slow earth flows in plastic soils are noted for occasional surges to rates of several meters per minute (Keefer and Johnson, 1983; Baum, 2003) or reactivation with sustained movement that results in many meters of displacement (Fleming et al., 1988). On the other hand quick-clay landslides usually move rapidly and retrogress a distance that is many times the slope height. Landslides on steep slopes (>25°-30°) always have the potential for rapid movement.

It has also been observed that debris flows commonly form on the toes of large landslide deposits (Reid, M.E., USGS, oral communication, 2003). Thus, material from a large slow-moving landslide may become part of a smaller rapidly moving landslide. Although such catastrophic movements may be relatively uncommon, nearly all slow landslides have the potential for abrupt or sustained relatively rapid movements ranging from a few decimeters to several meters or more. These guidelines do not recognize any set of circumstances, conditions, or characteristics that positively ensure that a slow moving or inactive landslide will not undergo future displacements large and rapid enough to damage or rupture a pipeline.

2.5 Methods of Estimating Displacement

Computation of displacements is of interest in application to pipeline engineering, because expected displacement determines the potential for slope movement to damage a pipeline. Accuracy of displacement predictions depends on many factors, including material properties, moisture conditions, geological details, and modeling details.

2.5.1 Estimating Total Displacement

When mapping landslides, it is usually desirable to estimate total past displacement. Such estimates often provide an upper bound on possible future displacements (Skempton et al., 1989). Major reactivation of large old landslide deposits, such as the Manti, Utah landslide (Fleming et al. 1988) and the Thistle landslide (Schuster and Fleming, 1986), which has resulted in major movement and enlargement beyond the previous boundaries of these landslides are notable exceptions. The most direct method is to measure offsets at the boundaries. For example, where a fence or road crosses a landslide the offset across the boundary gives an estimate of the total displacement. Displacement varies from point to point making the collection of multiple measurements of offset necessary to gain a sound picture of past landslide displacement patterns. Measurements of displaced volume usually do not provide reliable estimates of net displacement, because they are based upon vertical changes and do not track reference points on the surface.

Further, it is necessary to calibrate continuum models against reasonably well known landslide or ground movement records prior to using them to make predictions about displacements under various pore pressure or loading scenarios. Although not normally used in regional assessments, a geotechnical model is available to calculate the amount of deformation a slope is likely to undergo when subjected to earthquake ground motion. Among the input requirements for the model is an acceleration time history record.

Several empirical and semi-analytic approaches have been developed for predicting travel distance or potential travel distance of debris flows and rapidly moving landslides. For application to pipeline engineering, these methods have application to identifying areas where aboveground pipeline facilities might be subject to damage or inundation by debris flows as well as identifying where debris flows might impact existing deep-seated landslides and increase their potential for reactivation. Hungr et al. (2005) summarize most of the available methods, including their advantages and limitations. The majority of these methods require some type of observational data relating travel distance to one or more other parameters such as slope height, slope angle, or landslide volume.

The most reliable methods of estimating runout still rely on the presence of previous deposits. Precise prediction or estimation of displacement of coherent landslides or the runout of mobilized fluid landslides is not within the current capabilities of modeling methods. Models of granular or particle flow have been used to attempt to match the distances and paths of debris flows (Hungr and Morgenstern, 1984; Denlinger and Iverson, 2004; Iverson et al., 2004). However, no current models accurately model runout distances except in uniform materials that contain few irregular particles. Trees and other types of vegetation that commonly become incorporated in debris flows are irregularities that cannot be modeled successfully by these methods but can impart considerable influence on runout distances and flow paths.

Despite the advances being made in the modeling of landslide displacement and runout, it is still beyond reliable modeling capabilities to precisely estimate the velocity or the total displacement of dislocated earth materials. Recent work using calibrated models has provided displacement estimates within a factor of 2 or 3 of observed displacements.

2.5.2 Prediction of Time to Failure

In several case studies of creeping slopes that subsequently failed, post-failure empirical analysis of accelerating displacements has been relatively successful in predicting the observed time of failure. Small but measurable displacements commonly occur prior to initial failure of a slope or reactivation of landslide deposit. Saito (1965) observed an accelerating trend in these displacements and proposed using this trend in forecasting time to failure. Fukuzono, (1990) found that plotting inverse velocity (v^{-1}) versus time yields a straight line that can be used to predict the time of failure. Other workers have used Saito's observation or further developed the related theory (Voight, 1988; Kilburn and Petley, 2003). Recent work indicates that linearity probably is associated with crack growth and would be expected where brittle failure is the primary process occurring at depth. An asymptotic trend in the plot of inverse velocity versus time is expected where ductile failure or sliding on existing surfaces is occurring at depth (Kilburn and Petley, 2003). Using displacement observations at multiple points on the surface of the developing landslide or reactivation makes it possible to observe the spatial progression of the failure and greatly aids interpretation of the failure process (Petley, 2004).

2.5.3 Movement thresholds

Landslide movement thresholds, based upon either rainfall or pore pressure/water level, have been determined by comparison between measurements of rainfall or pore pressure and displacement. Such models are useful for making predictions and have modest data requirements, such as several years of displacement and rainfall observations. However, they are unable to predict changes in displacement by other factors, such as loading at the head of the landslide by debris flows or rockfalls or erosion at the toe. Further, they are limited by the degree to which observational data are representative of future extreme events. Grivas et al. (1996a, 1996b) and O'Neil et al. (1996) developed empirical models for predicting movement of a slow landslide based upon monthly rainfall. Rainfall thresholds have been developed for debris flow initiation for many areas of the world (Wieczorek and Glade, 2005). Pore pressure thresholds have been developed for individual landslides, such as the Johnson Creek landslide on the Oregon coast (Ellis et al., 2007).

2.5.4 Note on Effects of Seismic Events

Seismically triggered landslides are excluded from consideration in these guidelines. However, readers should be aware of guidelines developed by the California Geological Survey for evaluating and mitigating seismically induced landslides. The California guidelines are referenced here because they identify a threshold condition for initiation of sliding and use a procedure to estimate the amount of displacement. Both of these concepts would be valuable research topics for landslides triggered by precipitation, groundwater rise, stream erosion, or other non-seismic mechanisms.

The procedures described in the California Geological Survey guidelines include a pseudo-static analysis and a Newmark-type sliding block analysis. The pseudo-static analysis is used to define a threshold-of-motion acceleration value (yield acceleration) for the topographic and geotechnical conditions. The Newmark analysis is used to estimate the amount of downslope displacement that might be expected for that part of a design earthquake acceleration time history that exceeds the yield acceleration. Blake et al. (2002) developed recommended procedures for implementing the California guidelines.

A potential landslide condition that may be difficult to recognize is one in which earthquake shaking does not trigger a landslide, but produces ground cracks which make the slope more susceptible to rapid infiltration and development of hydrostatic pressures in the cracks. Such slopes may appear to be stable during heavy precipitation events and following very wet seasons. These slopes may have remained stable when subjected to earthquake shaking. An earthquake positioned in a location that produces ground motion in a direction that can induce minor ground cracks, followed by a moderate storm that occurs before the cracks can become filled in, could contribute to slope movement without being the triggering event.

2.6 Methods of Stability, Stress, and Deformation Analysis

Mathematical analysis of landslides is used to understand their individual mechanisms and make predictions about their responses to natural or human-induced changes in their geometry, external loading, ground-water levels, and other factors. Simple empirical methods of analysis use observational data to make predictions about time to failure, travel distance for debris flows, or threshold pore pressure levels or rainfall amounts to induce landslide movement. Limit-equilibrium slope-stability analysis is very useful in determining the factors that affect stability of a landslide mass, hillside, or earthwork as well as planning and evaluating potential remedial works for landslides. Limit-equilibrium methods have some important limitations because as their name implies, limit-equilibrium methods attempt to solve only the equations of equilibrium. Analytical solutions have been obtained for a few boundary and initial value problems that describe important landslide processes in one or two dimensions; however, their value is primarily for understanding the process rather than application to particular landslides. Numerical methods of stress and deformation analysis solve the equations of motion (or equilibrium) and continuity for different material constitutive models to provide estimates of deformation as well as internal forces/stresses. Some recent methods are even capable of analyzing the dynamics of large deformations associated with movement of rock slides and debris avalanches (Crosta et al., 2003; Denlinger and Iverson, 2004).

The following sections summarize the capabilities, data requirements, recent developments and trends, and pitfalls of the various types of mechanical analysis available for landslides. The discussion focuses on groups and classes of methods used in these analyses, rather than details of specific methods and software, which are constantly changing.

2.6.1 Limit Equilibrium Analysis

Despite considerable advances in numerical methods of stress and deformation analysis, review of recent literature reveals that limit-equilibrium methods continue to be used routinely in engineering investigations of landslides (Sharma, 2007). Research to improve the efficiency of limit-equilibrium methods also continues (Zhu et al., 2005). Limit-equilibrium methods require the analyst to make certain assumptions about the failure mechanism and provide no information about potential landslide displacement or deformation, because these methods attempt to solve only the equations of equilibrium.

Two-Dimensional Methods

Many limit equilibrium methods subdivide the landslide into vertical slices, as shown in Figure 2.14. An example of more complex slope stability analysis using methods of slices within landslides is shown by Wu (1969).

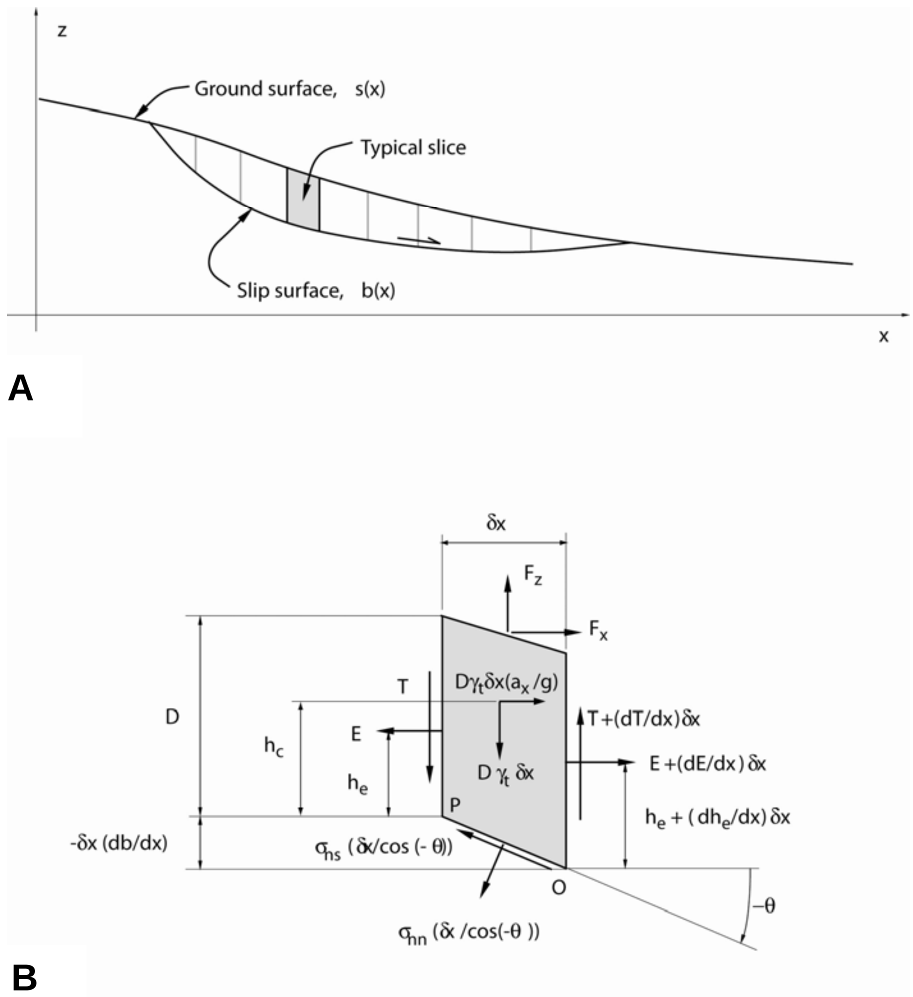


Figure 2.14 The method of slices used in limit equilibrium slope stability analysis (Baum, 2000, (A) Diagrammatic cross section of landslide, (B) detail showing slice dimensions and the forces and stresses acting on a slice)

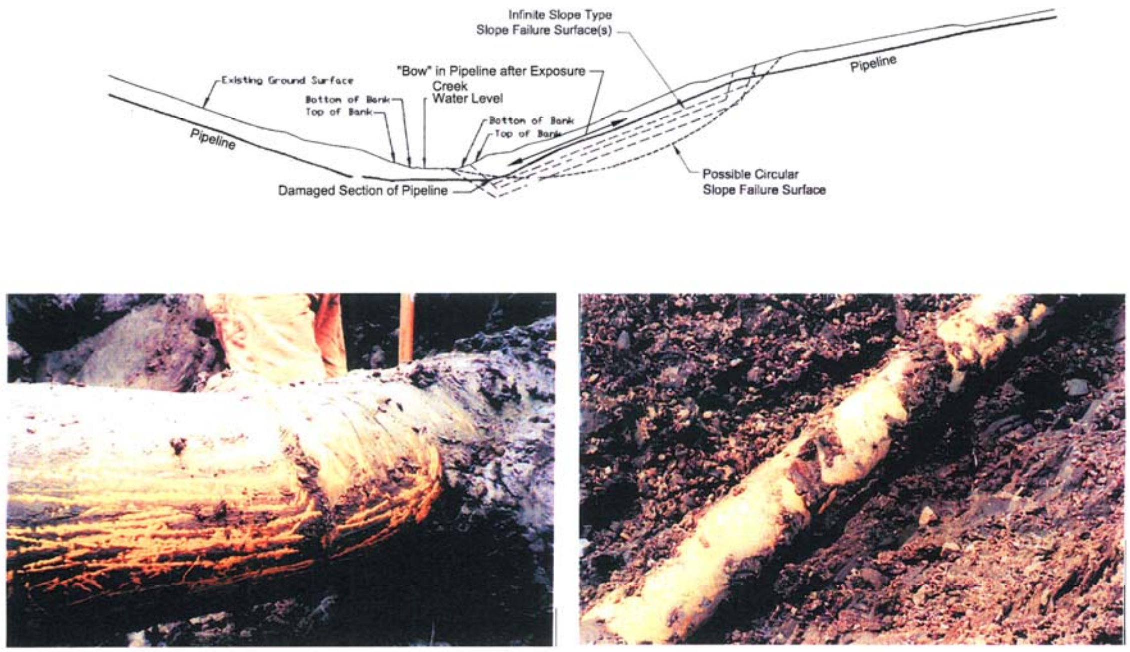
Many technical factors contribute to slope stability analysis including, but not limited to, geologic materials, slope angle, groundwater infiltration and levels, types and depths of vegetation, and daily and seasonal precipitation. Different approaches to and methods of stability analysis are relevant to different types and depths of landslides, e.g. shallow debris flows and deep rock slides, depending on their geometry, mode of failure, and other factors.

Methods of static slope stability investigations using available limit equilibrium analyses, including an assessment of their accuracy were presented in Duncan (1996) and chapter 9 of Blake et al. (2002). The primary differences between methods are how interslice forces are treated and whether the method satisfies all equations of moment and force equilibrium. Methods that satisfy all equations of equilibrium are considered more accurate than those that satisfy only some of the equations. (Duncan, 1996).

Errors in stability analysis arise more from choice of strength parameters, pore pressures and external loading than from the method of analysis used. Physically determining the detailed orientation of the geologic materials and variability of the strength and moisture at different depths of materials can affect the accuracy of slope stability analysis. Fracturing, groundwater movement, and other changes at a landslide site, make it difficult or impossible to determine slope stability conditions that existed prior to movement, even though landslide boundaries and depth can be determined precisely. Conversely, although the materials can be measured and groundwater can be monitored before a landslide failure initiates, it is difficult to determine the exact location of the slip surface prior to movement (see postulated failure surfaces for slide failure in Figure 2.15). Unless the shape of the slip surface is known in advance, limit-equilibrium methods must search for the so called "critical" slip surface, which has the lowest factor-of-safety. Most early search routines were limited to circular slip surfaces or fairly simple non-circular slip surfaces. Newer methods based upon optimization techniques of variational calculus (Baker, 2005) and neural networks (Samui and Kumar, 2006) as well as an approach using "stress acceptability criteria" (Sarma and Tan, 2006) have been devised to find the critical slip surface and overcome limitations of previous methods. These new methods can search for circular and noncircular critical slip surfaces in homogeneous and non-homogeneous materials and their success is not limited by user assumptions about the shape or general location of the slip surface.

Three-Dimensional Methods

Although available since the 1980s, the use of three-dimensional slope stability analysis is rare but becoming more common (Bromhead, 2004). These methods involve subdividing the landslide or potential landslide into vertical columns and solving depth-averaged equations of equilibrium to obtain a factor-of-safety. Three-dimensional methods differ from one another in the assumptions made about intercolumn forces and each is based upon a corresponding two-dimensional method. Two-dimensional analyses continue to be the most widely used methods of slope stability analysis, in part because they are widely considered to be more conservative than three-dimensional methods and in part because data and computational requirements of three-dimensional analysis are much greater than for corresponding two-dimensional analysis.



Looking at damaged section of pipeline.
Note: - Compression wrinkle or crushed zone over top half of pipeline.

Looking at "bow" in exposed pipeline.
Note: - Pipeline has lifted off clay till subgrade at bow.

Figure 2.15 Pipeline damage from landslide movement parallel to a pipeline and inferred slide failure planes used to define extent of landslide hazard along the pipeline alignment

Bromhead et al. (2002) and Bromhead (2004) reviewed circumstances where three-dimensional analysis is required; these include landslides of irregular shape, localized loading, weak zones, localized pore-water or pore-pressure concentration, and landslides with bedding controlled basal shear surfaces. In one recent application of three-dimensional methods, a digital elevation model is searched for the most critical slip surface centered at each grid cell to identify the areas most prone to rotational or other deep-seated failure (Brien and Reid, 2001; Brien and Reid, 2007). Bromhead (2004) described a numerically integrated "wide-column" approach that uses concepts from finite-element analysis to adapt methods of columns to landslides of general shape.

Authors of different methods of three-dimensional slope stability analysis have tested and verified their methods against corresponding two-dimensional methods and some simple three-dimensional cases (Chen and Chameau, 1982; Hungr et al., 1989). Such comparisons indicate that three-dimensional methods appear to give accurate results, but there are limited examples of rigorous evaluations of three-dimensional limit-equilibrium methods. Some recent work has shown that factor-of-safety from a three-dimensional analysis can be lower than that obtained from a two-dimensional analysis (Bromhead, 2004).

2.6.2 Continuum Stress and Deformation Analysis

Many landslides and hillsides can be modeled using a continuum approach. Numerical continuum methods currently available for slope stability and deformation analysis make it possible to compute a full solution of the stress and deformation equations in two and three dimensions (Hungr et al., 2005; Griffiths and Marquez, in press). Some models are specifically designed for modeling stress and deformation of coherent landslides and can also compute a factor-of-safety just as computed by limit-equilibrium methods (Savage et al., 2003; Smith and Griffiths, 2004). In a recent development (Cala et al., 2004) the shear-strength reduction technique used for computing the factor-of-safety has been extended to analyze several potential failure modes and slip surfaces. Besides computing factors-of-safety, some numerical codes are designed for modeling the movement of large, long-runout landslides, flows, and avalanches once initial failure has occurred (Crosta et al., 2003; Denlinger and Iverson, 2004; Hungr et al., 2005). Despite recent advances in software and methods, stress and deformation analyses of landslides still are not routine. They require more time and effort than limit equilibrium analyses and require considerable expertise to achieve accurate and meaningful results.

Numerical analyses currently available are based upon finite-element, boundary-element, and finite-difference formulations and are capable of handling a wide variety of constitutive models. This includes the ability to introduce strain-softening properties for the various materials, without the need to pre-define or “average” strength reductions. Strain softening has been implemented in various two and three-dimensional continuum codes. For example, Puebla et al. (2006) modeled progressive/retrogressive strain softening and resulting moderate to large (1 m or more) deformations in good agreement with observed ground movements. Troncone (2005) used a strain softening constitutive model to analyze the progressive failure mechanism of the slope and obtained good agreement between the observed and predicted failure geometry and mechanism. Some codes offer the possibility of coupled analyses of ground-water flow and slope deformation (Hungr et al., 2005). For example, Konietzky et al. (2004) modeled ground-water flow and slope stability in response to rainfall and changes in reservoir level. Most methods, including the traditional finite element analyses use Lagrangian (material) coordinates and are able to model slope deformation so long as it does not drastically distort the mesh (Hungr et al., 2005). Some recent efforts have used combined Lagrangian and Eulerian (fixed in space) coordinates to enable them to model large deformations without distorting the finite element mesh (Crosta et al., 2003). Other recent advances include zero-thickness elements for representing joints (Hürlimann et al., 2004) and fracture-mechanics elements to represent growth of fractures or slip surfaces (Zi and Belytschko, 2003).

One of the advantages of continuum methods over limit-equilibrium methods is that the model automatically or “naturally” determines the critical failure surface as part of the solution (Griffiths and Lane, 1999). Strain localization and development of a displacement or velocity discontinuity clearly delimits the depth of the failure zone (Figure 2.16). Numerical analysis of some complex landslides has predicted shear zones that are in close agreement with observed ones (Savage et al., 2000; Baron et al., 2005; Chugh et al., 2007). Another advantage of these methods over the limit-equilibrium methods is that no assumptions need to be made about interslice or intercolumn forces to compute the factor-of-safety. When there are sufficient data to characterize the spatial variability of subsurface soils (typically only for slopes in which failure results in consequences much greater than those of pipeline rupture), the ability to identify the most critical slip surface can be further enhanced using random finite element techniques (Griffiths and Fenton 1993, Fenton and Griffiths 1993)

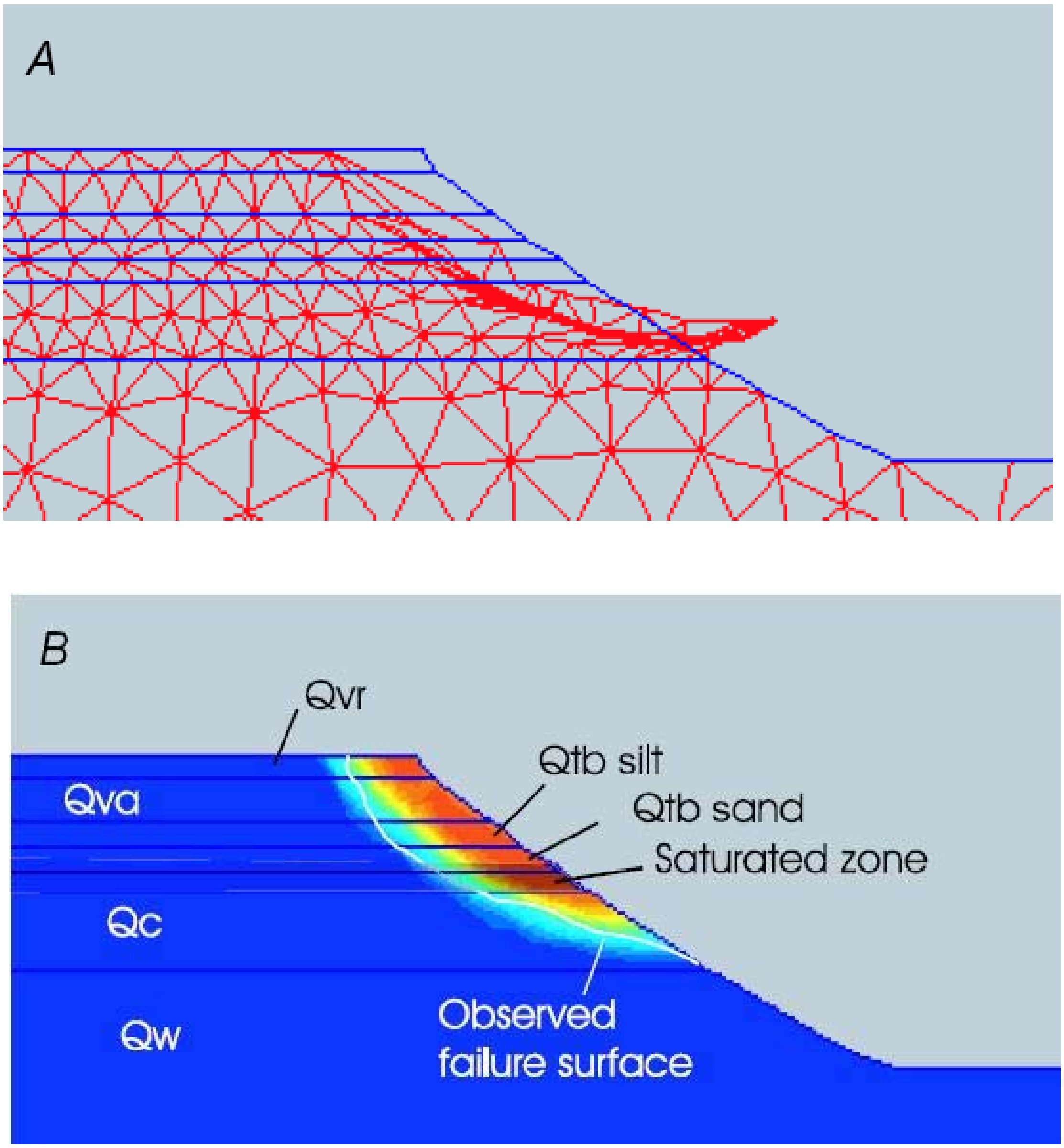


Figure 2.16 Example of results from finite element analysis (Savage et al.,2000), (A) deformed finite-element mesh showing computed displacements, (B) contour diagram comparing displacements to actual failure surface profile, warm colors correspond to larger displacements.

Rather than treating a hillside as an ideal continuum, the discrete or distinct element method subdivides the hillside into independent, disconnected pieces or elements. This method is well suited to slopes where discontinuities, such as faults, joints, bedding planes or foliation control the mechanism of instability. Thus, continuum methods that rely upon discrete element analyses have obvious applications to modeling the stability of rock slopes including failure as rock fall, rock slides, and debris avalanches, but it can also be applied to more coherent landslides (Hungr et al., 2005). A physical analogy for a two dimensional discrete element model is a stack of rods (of any desired cross sectional shape) with their long axes parallel to one another. In three dimensions, the discrete elements represent a pile of balls or blocks. The elements can be either rigid or deformable, but most methods allow the blocks to deform. Discrete element models apply the laws of dynamics to each individual element and its interactions with its neighbors and the surroundings to compute the forces acting on and within the mass as well as computing movement of the individual parts.

Rockfall simulation models are one of the simplest examples of discrete element analysis. For example, the extent of the areas potentially subject to rock fall hazards in the Yosemite Valley were obtained using STONE (Guzzetti et al., 2002), a physically-based rock fall simulation computer program. The software computes three-dimensional rock fall trajectories starting from a digital elevation model (DEM), the location of rock fall release points, and maps of the dynamic rolling friction coefficient and of the coefficients of normal and tangential energy restitution. For each DEM cell the software calculates the number of rock falls passing through the cell, the maximum rock fall velocity and the maximum flying height. However, the modeling software STONE is unable to consider the volume and mass of the falling boulder, the shape of the block, or the tendency of rock falls to split during successive impacts.

Although generally utilized when more detailed topography or profiles, with slope breaks, are available, the Colorado Rockfall Simulation Program (Jones et al., 2000) and a probabilistic rockfall program, based upon the work of Stevens (1998) incorporate shape and mass or just mass (Stevens, 1998) and provide data on both the velocity and impact energy of the rock fragment considered. The program by Stevens (1998) also provides estimates or rockfall runout for the various sizes (masses) considered.

In a survey of recent literature, Hungr et al. (2005) found that distinct element analysis has been used to investigate a wide variety of rock slope failure mechanisms including planar sliding, complex deep-seated sliding, rotation, toppling and buckling. Konietzky et al. (2004) used particle modeling to investigate creeping rock/ debris slide deposits on the slope of a reservoir and were able to decipher the failure mechanism and estimate the volume of material that would flow into the reservoir under worst-case conditions. Stead et al. (2004) used a hybrid finite element and discrete element method to model a complex rock slope failure mechanism. Coupled hydromechanical, distinct element modeling has been used to analyze the effects of drainage on landslide movement (Hungr et al., 2005).

2.7 Cautions Regarding Slope Stability Analyses

As discussed previously, slope stability calculations play a major role in evaluating existing slopes and are essential for assessing the effectiveness of geotechnical measures to reduce slope movement hazard. Typical practice is to establish safety factors for slope stability based upon experience and the relative importance of the slope in question.

Where practical, probabilistic determination of slope stability is generally preferred in order to provide a means to explicitly account for the numerous sources of uncertainty in slope stability calculations. The primary advantage of relying upon a probabilistic framework for expressing the required factor of safety is that the approach is much more amenable to being reviewed by others. The probabilistic framework provides a means to explicitly identify key areas of uncertainty and a numerical framework for expressing the effects of this uncertainty on the reliability of slope stabilization methods employed.

A variety of methods are available to compute slope stability in a probabilistic manner (e.g., El-Ramly et al., 2002, Haneberg, 2004, Nadim et al., 2005, Griffiths and Fenton, 1993), as well as Lee and Jones (2004) which includes several illustrative examples. Key differences in approaches typically fall into one of the following topics:

- Critical Slip Surface: Differences in the identification of the critical failure surface are primarily dependent upon the methodology used. This is particularly important where more critical non-circular failure surfaces can be identified. Where sufficient information exists to characterize the subsurface characteristics, finite element methods may provide a superior method to identify the most critical failure surface.
- Defining Parameter Variability: Differences in how variability is defined for key parameters, such as the soil strength and phreatic surface location, will impact the variability in computed slope stability factor. Where simplified closed form solutions for slope stability are utilized, for example for infinite slope solutions, the probability distributions are often selected for mathematical convenience (e.g., normal or lognormal). Approaches have been used to account for parameter variability include event trees, first-order second-moment approximations, first-order reliability methods, and Monte Carlo simulations.
- Spatial Correlation of Soil Parameters: Several researchers (e.g., El-Ramly et al., 2002, Griffiths and Fenton, 2004) have identified potential short-comings associated with neglecting the degree to which spatial correlation of soil parameters exist within a slope. The primary obstacle to accounting for potential spatial variability is the large amount of data required. At present, the information gathered in a typical site investigation program would likely be inadequate for an analytical assessment of the level of correlation.

As the selection of a particular approach for determining a probabilistic estimate of slope stability is largely dependent upon the information available, there will almost always be a related issue of how much information needs to be collected. As the amount of information will never be "complete" the decisions on what data needs to collected and the methodology employed to assess slope stability will always requires considerable judgment.

The task of implementing probabilistic assessment of slope stability is made somewhat easier by the fact that probabilistic analyses can be readily carried out in commercial software (e.g. SLOPE/W, Slide 5.0). These software applications require a definition of the probability distributions, based upon data or judgment, for key variables in the analysis. The random finite element approach described in Griffiths and Fenton (1993) and Fenton and Griffiths (1993) extends probabilistic techniques to include consideration of spatial variability of soil parameters in determining a probability distribution for slope stability. Considerable caution is necessary to avoid placing too much emphasis on the numerical values resulting from a probabilistic formulation of slope stability. In addition, it is important to recognize that a probabilistic definition of slope stability is only an indicator of the susceptibility of the slope in question to potential movement given a change in conditions (e.g., a triggering event such as extreme rainfall, earthquake ground shaking, or erosion of the toe of a slope).

2.8 Data Requirements

All types of landslide analysis have several basic data requirements in common. Table 2.2 summarizes the data requirements for different methods of analysis. At a minimum, data requirements include geometry of the landslide or potential landslide mass (surface topography, stratigraphy, structure and discontinuities, geometry of the basal slip surface) relevant strength properties, and subsurface water pressures. Stress and deformation analysis require some additional types of data beyond that required for limit-equilibrium methods (observed displacements, rates of movement, and elastic properties), but this addition is fairly modest. However, determining representative deformation moduli beyond the elastic range, necessary for continuum analyses of permanent ground movements and failures, is extremely difficult using normal laboratory or even field testing. The dimensionality and complexity of a slope or landslide have a great impact on the amount of data required to perform a satisfactory analysis regardless of the chosen method of analysis.

2.9 Challenges, Dilemmas and Reliability

During the last decade, several advances have been made in numerical modeling techniques for landslides. These include some incremental improvements to application of limit-equilibrium analyses, major advances in continuum and discrete element methods and increased use of probabilistic or reliability (Duncan, 2000) methods to define the degree of uncertainty in the results of numerical analyses of all types. These advances allow models to better represent and predict failure modes and mechanisms, slip surface geometry, displacement, and interaction between subsurface water and hillslope materials (Hungr et al., 2005).

Software for nearly all of the numerical modeling techniques described here (except new or experimental techniques) is available in ready-to-use commercial packages but some is also available in the form of published source code (e.g. Smith and Griffiths, 2004). The majority of the published analyses in recent years (including most of those cited in this report) have used commercial software packages to conduct sophisticated numerical analyses. Commercial packages generally offer technical support and software maintenance, but users are not able to view or modify the source code; thus the details of how the software handles certain situations or computations may not be understood unless it is well documented. Open-source software on the other hand is open to scrutiny and modification by the user, but generally little or no technical support is offered.

A wide variety of methods are now available for modeling landslides and slope stability and the widespread availability of powerful personal computers and modeling software make modeling adaptable to diverse project requirements and relatively easy and accessible. However, this abundance of user-friendly modeling tools opens the possibility of modeling without having any real physical understanding of the processes and factors involved

Table 2.2 Data types used in various numerical and analytical methods of landslide analysis

Data type	Source and purpose	LE	FE/FD	DE
Surface topography and landslide features	Detailed site survey (profiles or map) to define free surface of landslide as well as any surcharge loads, Detailed geologic mapping define plan-view shape of landslide and any shear zones or other features that subdivide the landslide into mechanically distinct parts	A	A	A
Site stratigraphy	Geologic mapping and subsurface exploration (drilling, trenching) define internal geometry of hillside or landslide, with attention to details that affect variation in material properties	A	A	A
Discontinuities and structure	Geologic mapping and subsurface exploration define internal geometry of hillside or landslide, with attention to details that affect bulk material properties and identify potential surfaces of shear or separation.	A	A	A
Slip surface geometry	Subsurface exploration and inclinometer or TDR monitoring defines the basal boundary of the landslide for accurate computation of the factor-of-safety by limit equilibrium methods or validation of predicted basal shear zone for other methods.	A	V	V
Shear strength parameters	Subsurface sampling and in-place or laboratory testing of landslide materials determines internal shearing resistance as well as shearing resistance along the basal slip surface	A	A	A
Stress-strain properties	Subsurface sampling and in-place or laboratory testing of landslide materials to determine stress-strain properties allows numerical models to predict landslide movement/deformation in response to changes in pore pressure, external loading or other factors.	N	A	A
Displacements and rates	Time series monitoring data of movement and pore pressure allow definition of minimum pore pressures needed to initiate or sustain movement. Spatial distribution of displacement amounts or rates provides check on model predictions.	A/V	V	V
Subsurface water pressures	Pore pressure at the basal slip surface and within the landslide as determined by monitoring (possibly supplemented by numerical modeling of ground-water flow) allows computation of the effective stress, which in turn affects strength and deformation of landslide materials.	A	A	A
Note: LE, limit equilibrium methods; FE/FD, finite element and finite difference methods; DE, discrete element methods; A, data used to perform analysis; V, data used to validate analysis results; N, not used by these method				

(Bromhead, 2004). Modeling efforts must be closely tied to field mapping and monitoring as well as laboratory and in-situ testing and measurements to ensure that meaningful results are obtained (Hungr et al., 2005). Modeling is a useful tool, but it must not be substituted for critical thinking and sound judgment.

A series of questions may help define some of the challenges and limitations of modeling.

- What are the objectives of modeling?
- What tools and how much modeling effort are needed to achieve the objectives?
- How much data of what quality is needed to produce the desired results?
- How far can the problem be simplified and how does one recognize adequate conceptual models?
- Are the modeling results consistent with field observations and measurements?

The first question, although obvious, is an important one, because modeling objectives must be clearly defined at the outset and the objectives must be suitable to determine the answers to the remaining questions. Modeling intended to make a predictions of landslide displacement over some given period of time requires different techniques, significantly more effort, and higher-quality data than modeling to determine a factor-of-safety. This translates into significantly higher costs, not only for the actual analysis, but also for the field investigation, monitoring, and material properties testing. In many cases the quality of the data available is only adequate to model present conditions and obtain some qualitative understanding of the failure mechanism and factors that influence stability. The question of simplification, although dependent on data quality, usually requires judgment and some experimentation to determine how much complexity must be included to enable the model to reproduce observed behavior of a landslide. Forward modeling to make predictions about potential movements or responses to various changes require much additional high-quality data (Hungr et al., 2005).

2.9.1 Cost

The cost of field data acquisition needed to support modeling is usually substantial. Costs specific to modeling include computer hardware and software and staff time needed to develop, parameterize, calibrate and run the model. The effort to perform empirical modeling and two-dimensional limit-equilibrium modeling is usually in the range of man days to man weeks per analysis case. Cost increases as a model becomes more complicated, by adding the third spatial dimension, time dependence, heterogeneity, or other special model features. As a result, the cost of using two-dimensional or three-dimensional, time-dependent continuum models to analyze displacements for complex landslides is likely to be five to more than ten times greater than the cost for empirical and limit equilibrium methods. Additional costs could be incurred to obtain soil samples and conduct laboratory tests necessary to define material property parameters for the analysis.

2.9.2 Reliability

The question of reliability in modeling landslides is challenging. Uncertainty remains in any model results and the input data are usually sources of much greater uncertainty than the method of computation (Bromhead, 2004).

Nadim et al. (2005) discussed uncertainty in terms of a probabilistic approach. Among other notable findings, they report that in some cases a potential slip surface with a higher factor-of-safety may also have a higher probability of failure than a slip surface with a low factor-of-safety. As illustrated in Figure 2.17, this can result from greater uncertainty in pore pressure, strength parameters and other factors.

Further, most probabilistic analyses lack the necessary data to compute an actual annual probability of failure; rather they provide a measure of the uncertainty in the input data and the computed factor-of-safety or displacement. Computing an annual probability of landslide movement or slope failure requires information about landslide recurrence obtained either by historical or geo-chronological methods described previously.

Probabilistic analyses require additional effort and some additional data (probability or frequency distribution of strength parameters, range of uncertainty on pore pressures, slip surface geometry, and so forth), but they complement conventional deterministic analyses (Nadim et al., 2005).

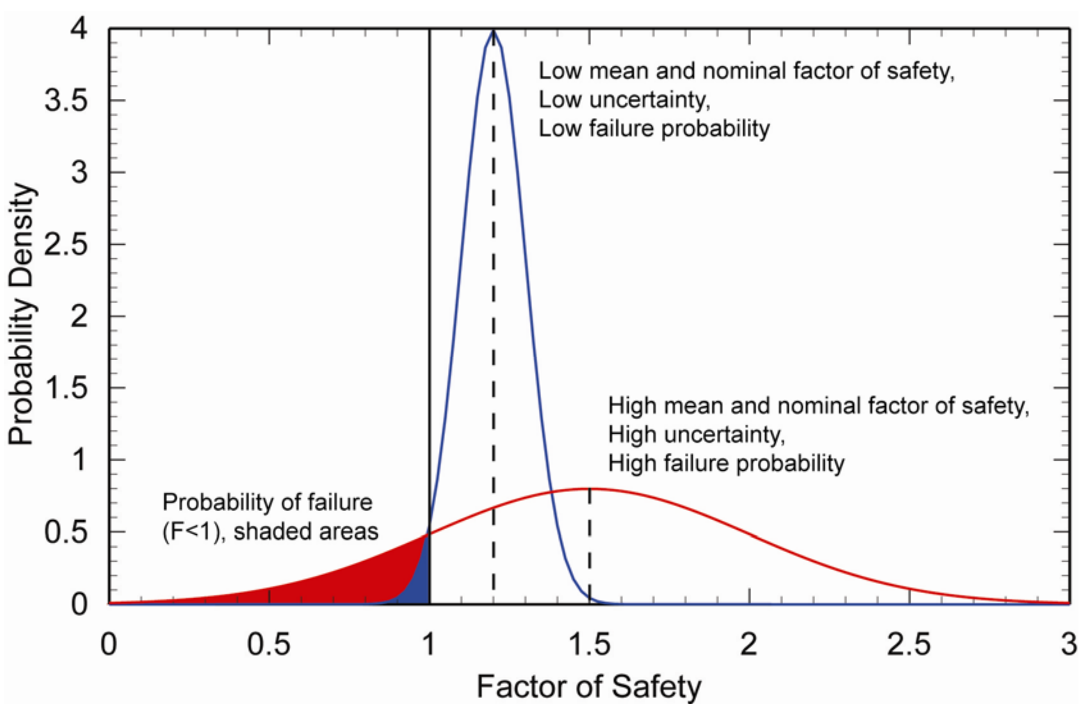


Figure 2.17 Probability chart for factor-of-safety, illustrating the relationship between data uncertainty, factor-of-safety, and likelihood of failure (after Nadim et al., 2005)

Interpreting computed displacements in a probabilistic sense has special challenges. Several sources of error contribute to the differences between computed and observed displacements:

- Measurement error in the observed displacements, which in most cases is relatively small
- Uncertainty in model parameters
- Uncertainty in the geometry of zones that have similar geotechnical properties and geologic structures
- Uncertainty and groundwater levels and pore pressures, and uncertainty or inaccuracy in the actual computational models

Natural soils and rock tend to be nonuniform and have more complex stress-deformation behavior than engineered materials (Duncan, 1996). Preexisting debris flows, rock falls, rock slides, slumps, and other types of landslides provide "ground-truth" data from which estimates of average, maximum, and minimum future landslide runouts can based with the confidence that these estimates included *in situ* conditions and irregularities of the real slopes under consideration. Therefore model construction, calibration, and analysis requires considerable time and effort. Reliability of calculated movements for natural slopes is not as great as for engineered embankments (Duncan, 1996).

Computing displacement for different scenarios of changing pore pressure, erosion, or changes in external loading is usually necessary to define the range of probable slope responses to future events. The task of defining the most probable displacement during the design life of a pipeline becomes an exercise in combining model uncertainty with the computed slope responses of the different scenarios and their likelihoods.

2.10 Subsurface Exploration

Drilling and trenching are the most commonly used methods for subsurface exploration of landslides. Geophysical techniques are sometimes used where drilling is not feasible or to aid extrapolating measurements from a single borehole or between boreholes. The most commonly used geophysical techniques include seismic reflection, seismic refraction, ground penetrating radar, and methods based upon electrical resistivity. In this section, a brief summary of commonly used techniques is presented and recent developments are highlighted. McGuffey et al. (1996) provide a complete detailed description of subsurface exploration techniques and methods of data presentation.

Drilling of deep-seated landslides is the most useful and most widely applicable method for subsurface exploration of landslides. Boring logs for existing water wells or oil and gas wells on or near a landslide can be useful sources of supplemental information but in many cases the descriptions are too general to be of specific value. Oil and gas wells have a higher chance of having downhole electric or nuclear logs, but the upper few tens to hundreds of feet is of little importance to petroleum resource companies and tends to be neglected. At many sites, focused drilling is necessary to determine the depth of the slip surface and the geometry of the landslide mass. For documenting landslides, drilling and sampling can determine types of subsurface geologic materials, locations, and orientations of joints, landslide fracture lines, and ground water levels. Although boring methods have changed little from those described by McGuffey et al. (1996) a few new techniques and improvements to old ones are worth noting.

Borehole logging methods including electric and nuclear logging have been described by McGuffey et al. (1996). These logging techniques help to characterize lithology and rock or soil density throughout the depth of borehole. Large diameter boreholes and downhole logging methods have been used as an informative method in landslide investigations, e.g. in Napa County, California by Johnson and Cole (2001). Borehole televiewers have recently been developed to help with logging of smaller diameter boreholes (Borchers, 1994; Nakamura, 2004).

For landslides involving rock masses, rock strength properties including friction angle of rock surfaces, the roughness of natural rock surfaces, fracture infilling, and recently displaced fractures are important parameters to be considered (Wyllie and Norrish, 1996). Much of this information can be obtained from boreholes. Use of oriented core sampling methods, with or without complementary borehole video or photographic examination provides high quality information on orientation of rock structure or bedding.

Direct-push (Geoprobe®) techniques have been developed for advancing small-diameter holes (up to 75 mm) in soil and soft rock by direct push or driving a sampler into the ground. This method returns high-quality samples for logging and some types of testing. Probing can reach depths of 30 m, which is adequate for many landslide investigations. Probing can be used as a primary method for subsurface exploration on smaller landslides or as a rapid method of creating additional holes for installing piezometers or other instruments (Bianchi and Farrington, 2001). At steep or difficult sites, probing can be performed by a crew of two or three using a gasoline-powered or electric-powered breaker hammer with a manual jack can be used to extract the sampler. Hydraulic probing rigs that mount on the back of a pickup truck as well as self-contained track-mounted rigs are also available. Tools for cone penetration testing (CPT) as well as electrical and hydraulic logging are available for these rigs.

CPT has been available for some time on larger truck- and track-mounted rigs and has evolved into a highly developed technique for geotechnical subsurface exploration, with many variants as described by McGuffey et al. (1996). A hollow steel rod with a conical tip is forced into the ground while the required force is recorded nearly continuously. Within soft to stiff or loose to compact soils, electronic CPT probes can be used to obtain continuous profiles of subsurface characteristics to depths of 30 m or more within a short time period. Since CPT methods typically record variations in subsurface conditions over a penetration distance of 50 mm or less, CPT profiling is effective in detecting the presence of weak or sheared zones that need further investigation by more direct methods, as well as the boundaries between or within various soil strata. Cone penetrometers can be equipped for geophysical and piezometric measurements. Thus, electronic CPT equipment is also typically capable of determining the variations in dynamic porewater pressures with depth, as well as porewater pressure dissipation properties. It can also be utilized to determine downhole shear wave measurements, or electrical resistivity profiles.

One of the most useful methods for investigation landslides is by the drilling and downhole logging of large-diameter borings (Johnson and Cole, 2001). Rotary bucket borings, commonly referred to as bucket auger borings, typically consist of a 24 to 36-inch diameter boring that is advanced 15-30 cm each run during drilling and can be advanced up to depths approaching 60 m in soft to firm rock types (Figure 2.18). A significant advantage of a bucket auger boring is the ability of the geologist to enter the boring in a metal safety cage for detailed logging of the *in situ* geologic conditions. Downhole logging provides a means to directly record bedding and joint orientations, and to observe and sample subsurface geologic materials including the basal rupture surface.

The downhole logging method is limited by the presence of excessive groundwater seepage and borehole stability and should not be performed where there is a potential for borehole caving, rockfalls, noxious gases or an oxygen deficient atmosphere. Many jurisdictions will have strict safety regulations for “confined space” work such as downhole logging. For example, the California Occupational Safety and Health Administration (CAL-OSHA) has imposed stringent safety requirements on downhole logging procedures including requirements that the geologist be present during drilling of the boring to observe borehole stability, ventilation of the boring during downhole logging, air monitoring requirements, the use of a 5-foot length surface casing, two-way communications and the use of a safety harness.

Figure 2.18   Preparing to downhole log a bucket auger boring

Acoustic televiewer tools must be used in fluid-filled holes, but the fluid can range from murky water to drilling mud. Acoustic televiewer data provide a virtual caliper profile of the borehole, as well as orientations of planar features, such as bedding and fracture planes, provided that the features are associated with relief on the borehole wall (see Figure 2.20).

Both televiewers use magnetometers for determining north orientation. The acoustic televiewer uses a rotating sonar transducer to send a signal which reflects from the borehole wall and back to a receiver on the tool. Travel time is used to map the distance to the borehole wall and reflectance intensity is used to define fracture zones and voids. The acoustic televiewer is very useful to capture an image of zones where core loss occurred. The dip direction and dip magnitude of rock structure can be mapped with this tool for subsequent analyses. Landslide masses can be difficult to differentiate. The optical and acoustic televiewers allow structure in the landslide mass to be documented for comparison to the rock structure below the landslide slip surface.

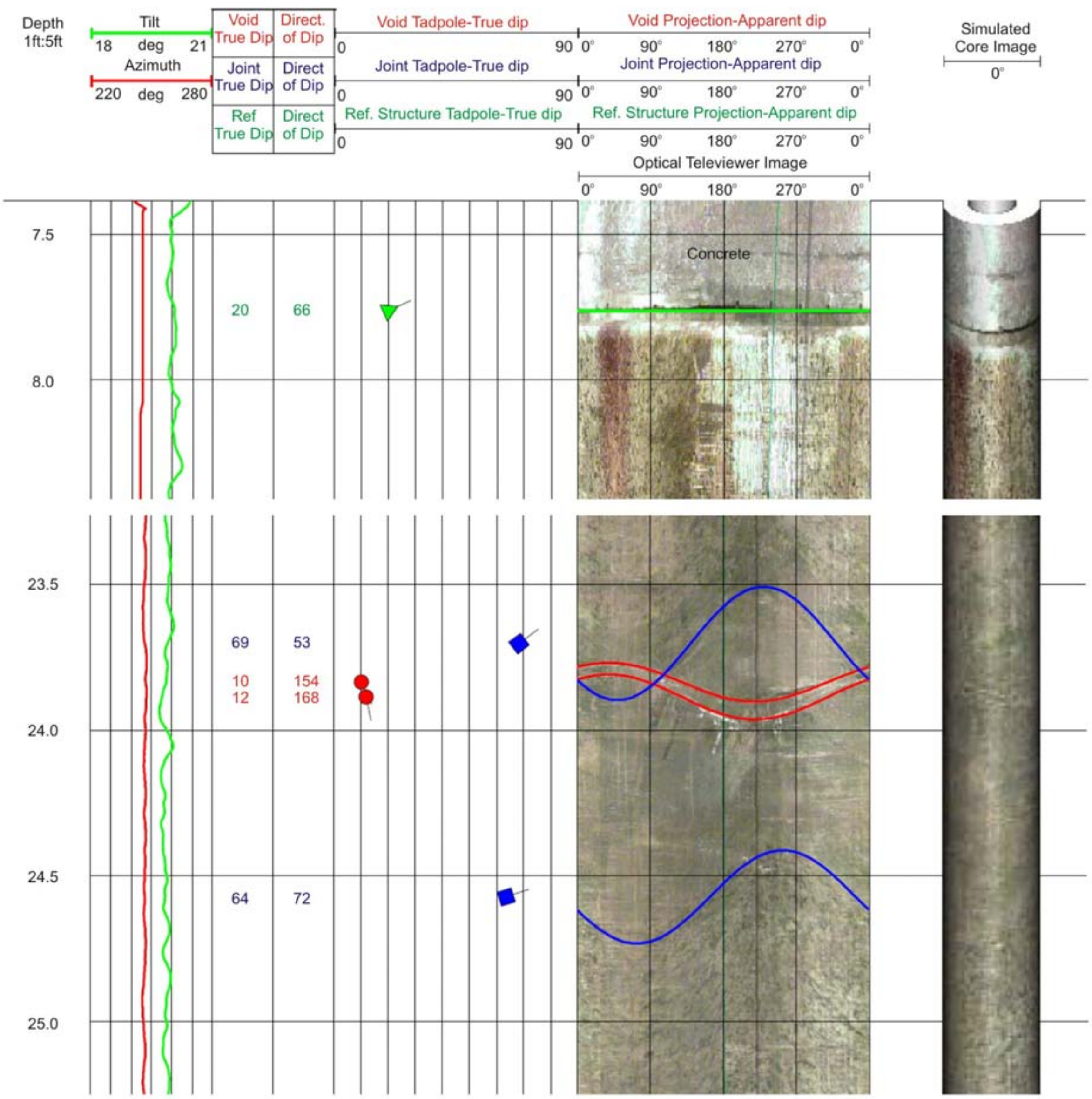


Figure 2.19 Example of optical televiewer virtual core with rock structure noted (modified from an image provided by Geovision Geophysical Services)

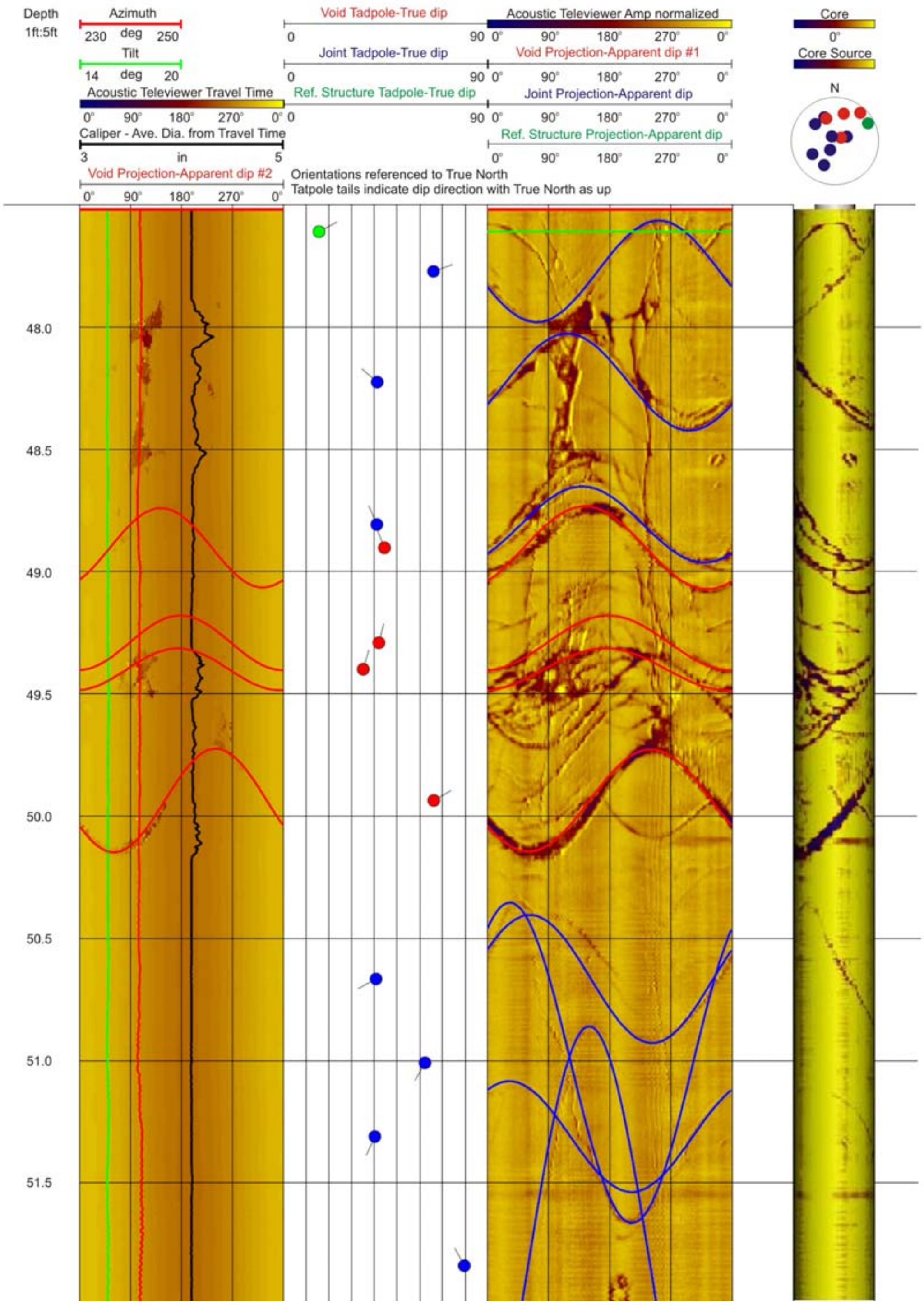


Figure 2.20 Example of acoustic reflection virtual core with rock structure noted (modified from an image provided by Geovision Geophysical Services)

Trenches and test pits permit direct observation and sample collection to depths of 3 m to 6 m with adequate shoring or stepped excavation in the case of trenches. Consequently, trenches and pits are useful in landslides of moderate depth, and near the edges of deep landslides. Trenches can be excavated in a few hours by backhoe at relatively low cost and provide a three-dimensional view that is unachievable by any other method. Information obtained by careful logging of trenches aids in accurate interpretation of samples and cuttings from boreholes. Trenches near the toes of landslides commonly expose the basal shear zone, which may be quite different in composition and appearance from most of the landslide debris, so that the shear zone materials can be adequately sampled and tested (Figure 2.21).

A

B

Figure 2.21 Examples of exploratory trenches to investigate landslides (A) Location of exploratory trenches in a landslide deposit on the southeast side of Snodgrass Mountain, near Crested Butte, Colorado, (B) Landslide basal shear zone exposed in lower trench (red paint spots added to highlight contact between gray crushed-shale landslide debris and yellow-brown sandy clay of shear zone material)

Use of test pits or trenches can be limited by land use or environmental restrictions, particularly at locations on public land managed by federal or state agencies that have specific regulations that require assessment of environmental impacts. Certain types or densities of vegetation can impede or prohibit access to locations for any type of subsurface investigation activities, particularly those that cause substantial ground disturbance. Use of test pits or test trenches and large diameter boreholes is also often limited by the presence of seepage or ground water levels, or materials having minimal stand-up time close to the ground surface. Sonic drilling techniques can be used to obtain near continuous disturbed cores of soils, ranging from fine sands and silts or clays to cobbles and boulders, and weak to moderate strength rock. Recent investigations in British Columbia indicate that such soils and rock can be penetrated to depths of the order of 40 m and possibly more (Butler, written comunication, 2007). Sonic drilling methods can also be used as a relatively rapid means to permit installation of slope inclinometers and conventional or rapid response piezometers.

The use of geophysical techniques in subsurface exploration is based upon attempts to relate changes in physical properties such as the elastic modulus or electrical resistivity to changes in lithology, the location of the water table, or other subsurface features of interest to landslide investigations. Ground penetrating radar can be used to detect the soil or rock strata and bedding features, and in particular, zones of disturbance to depths of the order of 5 m to 15 m in granular soils or rock, although it may have limited penetration within fine grained cohesive soils. Use of these indirect methods requires considerable skill in interpretation of the results. Example applications include locating the base of a landslide deposit by seismic reflection and refraction methods (Williams and Pratt, 1996; Corsini et al., 2006), or by a combination of seismic and electrical methods (Bogaard et al., 2000; Chelli et al., 2005). Electrical methods have also been used to investigate ground water distribution and landslides (Hiura et al., 2000). Geophysical techniques are often best combined with direct investigation methods to permit correlation and corrections to the inferred depths and properties of soil or rock strata while assisting in interpreting the variations in conditions between the direct investigation sites.

2.10.1 Sampling

Samples obtained from the ground surface or from subsurface exploration can be used to determine the types and strengths of the geologic materials involved in landslides. In some cases, samples can be used to determine the geologic age of the materials and possibly the previous age of landslide movement. As noted previously, radiometric analysis of wood or charcoal fragments found beneath a landslide can be used to determine the approximate age of previous historic/prehistoric landslide movement. Regardless of the specific tests or analyses planned for a particular sample or suite of samples, the object of sampling is to obtain materials that represent the properties or range of properties relevant to understanding past, present, and future behavior of the landslide. The heterogeneous nature and complex history of most landslides and landslide-prone areas make it imperative that the relationship of samples and sample locations to the overall geometry and structure of the landslide or potential landslide be well understood. Without adequate understanding it is very likely that irrelevant materials will be sampled and tested. For example, it has been observed in many relatively slow-moving landslides that the basal and lateral shear zones consist of much weaker materials than those that make up the main body of the landslide (Baum and Reid, 2000). Therefore, sampling and testing the shear-zone material is essential, whereas determining the strength of the landslide mass above the shear zone is less important in stability analyses.

Sample recovery is typically less than 100% but careful examination of recovered samples makes it possible to approximately reconstruct the distribution of materials in the subsurface. However, sample loss can lead to questions regarding the fidelity of the sampling program as weaker, fragmented, or easily washable materials that might exist in a shear zone will be more prone to sampling loss. In cohesive clay soils, it is sometimes possible to recover materials from the basal shear zone for shear strength testing; however it is usually necessary to use inclinometer observations to confirm the depth of sliding. In cases where the basal shear zone can be sampled directly, either in a trench or large diameter borehole, it is possible to obtain relatively undisturbed block samples for oriented (shearing parallel to the movement direction) direct shear tests of the slip surface.

2.10.2 Testing

The main purpose of testing landslide materials is to determine strength and hydraulic properties (Wu, 1996; Lambe and Whitman, 1969). Determination of shear strength of materials at the landslide slip surface is relevant to stability analysis, estimates of landslide movement and understanding failure mechanisms of slopes (Leroueil, 2001). Landslide materials are often inhomogeneous and the strength parameters can vary over an order of magnitude between different materials. Therefore correctly identifying slip-surface material is critical to obtaining representative test results. This can be a challenge especially for smaller or shallow landslides that are less likely to have a well-defined slip surface. In the case of potential landslide areas a detailed exploration sampling program is needed to identify and sample materials from potential slip surfaces. Hydraulic properties are used in predicting effects of rainfall, subsurface drainage and other factors on subsurface water pressures (Baum and Reid, 1995; Iverson, 2000; Hungr et al. 2005). Field tests usually provide the most meaningful values of hydraulic properties for landslide modeling purposes. Methods that have been utilized effectively to determine the ground water and piezometric conditions at depth within soil and rock, including “perched” or non-hydrostatic conditions, include multi-port piezometers, CPT dissipation testing and conventional falling/rising head testing (Butler, written communication, 2007; McGuffey et al., 1996).

Shear strength

Once relevant materials have been obtained for testing, consideration must be given to what type of tests that should be performed. Strength testing should attempt to duplicate field conditions as closely as possible. These conditions include the stress state, stress path, rate of shear, drainage, whether the material has been sheared previously, and whether displacement is concentrated on a discrete plane or distributed across a zone. A complete understanding of soil behavior in the context of slopes is needed to plan a soil testing program and interpret test results (Lerouiel, 2001). The stress-strain and strength properties of soils are typically determined in the laboratory by direct shear and triaxial tests. Some soils, such as coarse granular materials, and sensitive silty or organic soils present at depths of 10 m or more below the ground surface, and fractured or friable rocks may be difficult or impossible to sample without excessive disturbance. In such cases, use of field testing methods such as the Menard Pressuremeter, dilatometer and strain controlled field vane methods may be desirable or necessary to permit determination of strength values (McGuffey et al., 1996; Wu, 1996). Wu (1996) provides a complete description of laboratory and field test procedures.

Shear strength varies with displacement and soil porosity. Dense and cemented soils display a peak strength that is developed within the first few millimeters of displacement. Upon further shearing the soil weakens toward the so-called residual strength. Peak strength is usually considered relevant to first-time slides in natural normally consolidated clay and intact rock. The fully softened strength is relevant to first-time failure of stiff fissured clays and claystones (Skempton, 1985; Wu, 1996). Residual strength is generally considered relevant to reactivation of landslides (Skempton, 1985), but desiccation or precipitation of minerals from pore water can cause strength regain between episodes of movement (Bromhead, 2004). In some cases, residual strength also appears to be relevant to analyzing progressive failure (Dixon and Bromhead, 2002).

Recent improvements in test procedures

The main advancements in laboratory testing during the last decade have been development and improvement in stress-controlled testing and improved methods for unsaturated soils (Jotisankasa et al., 2007) whereas most traditional methods were strain controlled and restricted to saturated soils. Standardized procedures for tests based upon direct shear, torsional shear, and triaxial methods have long been codified in standards of the American Society for Testing and Materials (ASTM) and national institutes of standards of other countries. Perhaps the most notable recent improvement in these procedures is that automated equipment for conducting these tests is readily available from commercial sources; however the tests themselves have changed little over the years. Systems for static and dynamic, controlled stress, triaxial tests are also available commercially. Recently Sassa et al. (2004) developed a new torsional shear testing device that allows stress or strain control for static and dynamic testing. The primary advantage of stress controlled tests is their ability to mimic stress conditions within different parts of the landslide. Torsional shear testing also permits measurement of residual shear strength without repositioning of the sample for re-shearing as is required in direct shear tests.

Improvements for testing unsaturated soils include methods for determining soil water characteristics, methods for testing the shear strength of unsaturated soils, and a new framework for applying the effective stress concept to unsaturated soils. Research and development has continued on ways to improve laboratory systems for measuring shear strengths of unsaturated soils (Jotisankasa et al., 2007; Miller and Hamid, 2007). Recently Lu and Likos (2006) introduced the suction stress characteristic curve as a framework for extending the effective stress concept to unsaturated soils. This framework overcomes many of the limitations of previous attempts to describe the mechanical behavior of unsaturated soils (Bishop, 1959; Fredlund and Rahardjo, 1993).

3.0 SUBSIDENCE HAZARD

Land subsidence is a gradual settling or sudden sinking of the Earth's surface owing to subsurface movement of earth materials. Subsidence is a global problem and, in the United States, more than 44,000 km^2 in 45 States, an area roughly the size of New Hampshire and Vermont combined, have been directly affected by subsidence. The principal causes are subsurface fluid withdrawal, drainage leading to oxidation of organic soils, sinkholes, underground mining, hydrocompaction, thawing permafrost, and natural consolidation (National Research Council, 1991). More than 80 percent of the identified subsidence in the United States is a consequence of the exploitation of underground water, and the increasing development of land and water resources threatens to exacerbate existing land subsidence problems and initiate new ones (Galloway et al., 1999). In many areas of the arid Southwest, and in more humid areas underlain by soluble rocks such as limestone, gypsum, or salt, land subsidence is an often-overlooked consequence of land-use and water-use practices. Some subsidence also is associated with tectonic and volcanic processes; however this type of subsidence need not be considered a credible independent risk to pipelines.

Though subsidence generally refers to the downward motion of land surface, lateral ground movements accompany the subsidence. Both kinds of surface displacement affect pipelines but it is generally the lateral component that is responsible for greater damage as it can create large compressive forces in the pipeline and lead to upheaval buckling as illustrated in Figure 3.1. The lateral movements at land surface can be attributed to relatively deep-seated poroelastic deformation, to flexures or bending of the land surface and to ground failures such as those associated with the collapse of surficial material into underground voids (for example, sinkholes) and with differential subsidence or tensional stresses in the subsurface materials (for example, earth fissures).

Regional subsidence features (such as aquifer-system or reservoir compaction accompanying ground-water or oil and gas extraction) generally create relatively small lateral (sub-horizontal) strains at the land surface owing in part to poroelastic deformation of the aquifer system, and in part to the flexure of land surface. Poroelastic deformation refers to the coupled interaction between fluid flow and deformation of the skeletal-matrix of the host rock. Flexure characterizes the motion of the land surface subjected to the relative vertical displacement of some portion of the surface, such as the small component of lateral movement associated with the rotation or tilt (slope) of the land surface between two sites with differing amounts of subsidence.

Figure 3.1 Examples of upheaval buckling of gas pipelines resulting from subsidence related to oil extraction

Meters of vertical displacement may occur locally regardless of the type of process causing the subsidence. Identifying locations of future subsidence hazards relies upon (a) identifying areas that have subsided recently, (b) areas that are actively subsiding, (c) areas where activities leading to subsidence will occur, or (d) areas that are very similar in terms of topography, geomorphology, hydrology, and soil properties to areas where evidence of past subsidence and subsidence-related hazards is observed.

The following types of subsidence hazards are addressed in this document:

1. Subsurface fluid withdrawal: Deep-seated deformation of the porous skeletal matrix of the saturated rocks, owing to fluid-pressure declines principally caused by ground-water and hydrocarbon discharge and recharge. Two types of ground response are discussed:
    a. Aquifer-system compaction—principal hazard associated with ground-water mining in susceptible alluvial, basin-fill deposits
    b. Ground failures—earth fissures and surface faults associated with areas of differential ground displacements
2. Drainage of organic soils: Primarily oxidation of peat, muck, bog, fen, moor, and muskeg deposits associated with desiccation after water tables have been lowered to enable agricultural and other land uses.
3. Sinkholes: Typically localized collapse of the overburden into underlying cavities that form in relatively soluble deposits such as salt, gypsum, and carbonate rocks (for example limestone, dolomite). Two general types of sinkholes are discussed:
    a. Natural—karst terrain
    b. Anthropogenic—accelerated dissolution and collapse related to water-use, petroleum extraction and mining practices
4. Underground mining: Often gradual downwarping, sometime sudden collapse over mine footprints; typically associated with coal mines.
5. Hydrocompaction: Shallow subsidence associated with wetting of dry, low-density sediments.
6. Thawing permafrost: Subsidence associated with the development of thermokarst terrain.
7. Consolidation: Gradual reduction in volume and increase in density of a soil mass in response to increasing depositional load or change in effective stress conditions arising from natural processes or intentional surcharge loading related to construction activities, such as the construction of a highway embankment, site grading, or foundation improvements using preloading.

Piping and internal erosion is often associated with wetting or modification of natural or prior drainage conditions within erodible sediments such as loess, fine sands and silts, and plays a role in exposing earth fissures through erosion to fissure gullies. Piping and internal erosion within erodible sediments is not covered in this document. Earth fissures are covered in this document as indicated in item 1b above.

3.1 Recognition

The occurrence of land subsidence is most obvious in the case of catastrophic sinkholes such as those in the soil mantled karst of Winter Park, Florida (Figure 3.2). Where ground-water mining or drainage of organic soils is involved, the subsidence is typically gradual and widespread. In the absence of obvious clues such as protruding wellheads, failed well casings, broken pipelines, drainage reversals, and reduced freeboard in canals and aqueducts, repeat measurements of land-surface elevation are needed to reveal regional subsidence.

The problem of detection in regional land subsidence is compounded by the large areal scale of the elevation changes and the requirement for vertically stable survey bench marks located outside the area affected by subsidence. Where stable bench marks exist and repeat surveys are made, subsidence is easily measured using professional surveying instruments and methods.

Figure 3.2 Recognition of subsidence hazards (A) Cover-collapse sinkhole, Winter Park, Florida (1981), (B) Approximate location of maximum subsidence measured in the San Joaquin Valley, California (1977)

3.1.1 Known Subsidence Areas

Many subsidence areas in the United States have been identified, mapped, and documented. Most anthropogenic land subsidence in the United States is caused by the withdrawal of subsurface fluids from porous granular media, although humans also have caused widespread and significant subsidence by other processes. As measured by area affected, underground mining of coal and minerals, and drainage of organic soils are the most significant. Collectively, the impacts from these processes rival those from withdrawal of subsurface fluids. The National Research Council (1991) estimates that about 8,000 km^2 and 9,400 km^2 of land, respectively, have subsided because of mining and drainage of organic soils. Though mining subsidence is widespread and mostly associated with coal extraction, organic soil subsidence is concentrated in two areas, the Florida Everglades and the San Joaquin–Sacramento River delta, California (National Research Council, 1991; Galloway et al., 1999).

Subsurface Fluid Withdrawal

Withdrawal of subsurface fluids from clastic granular sediments has permanently lowered the elevation of about 26,000 km^2 of land in the contiguous United States (Figure 3.3), an area of similar extent to the state of Massachusetts (Holzer and Galloway, 2005). Permanent subsidence can occur when fluids (primarily water and hydrocarbons) stored beneath the Earth's surface are removed by pumpage or drainage. The reduction of fluid pressure in the pores and cracks of aquifer systems and petroleum reservoirs, especially in unconsolidated clastic rocks, is inevitably accompanied by some deformation of the aquifer system or reservoir. Because the granular structure, the "skeleton", of the fluid-bearing rocks is not rigid, a shift in the balance of support for the overlying material causes the skeleton to deform slightly. Almost all the permanent subsidence in aquifer systems is attributable to the compaction of aquitards during the typically slow process of aquitard drainage (Tolman and Poland, 1940). The primary problem associated with the subsidence was flooding, but there were also cases of structural damage to infrastructure, including pipelines, attributed to horizontal strains on the sides of the subsidence bowl.

Drainage of Organic Soils

Land subsidence invariably occurs when organic soils are drained for agriculture or other purposes. Causes include oxidation, compaction, desiccation, erosion by wind and water, and, in some cases, prescribed or accidental burning. The effects of compaction and desiccation after initial draining can be dramatic, because organic soils have very low density caused by their high water content (as much as 80-90 percent). The balance between accumulation and decomposition of organic material shifts dramatically where peat wetlands are drained.

Oxidation promotes a transformation from fibrous vegetative material to lower-volume organic decay products with the liberation of CO_2 and water vapor. Whereas natural rates of accumulation of organic soil are on the order of 10 cm per 100 years, the rate of loss of drained organic soil can be 100 times greater, as much as 10 cm/yr in extreme cases. Thus, deposits that have accumulated over many millennia can disappear over time scales that are very relevant to human activity. In general, the subsidence due to groundwater lowering is much larger than that due to oxidation.

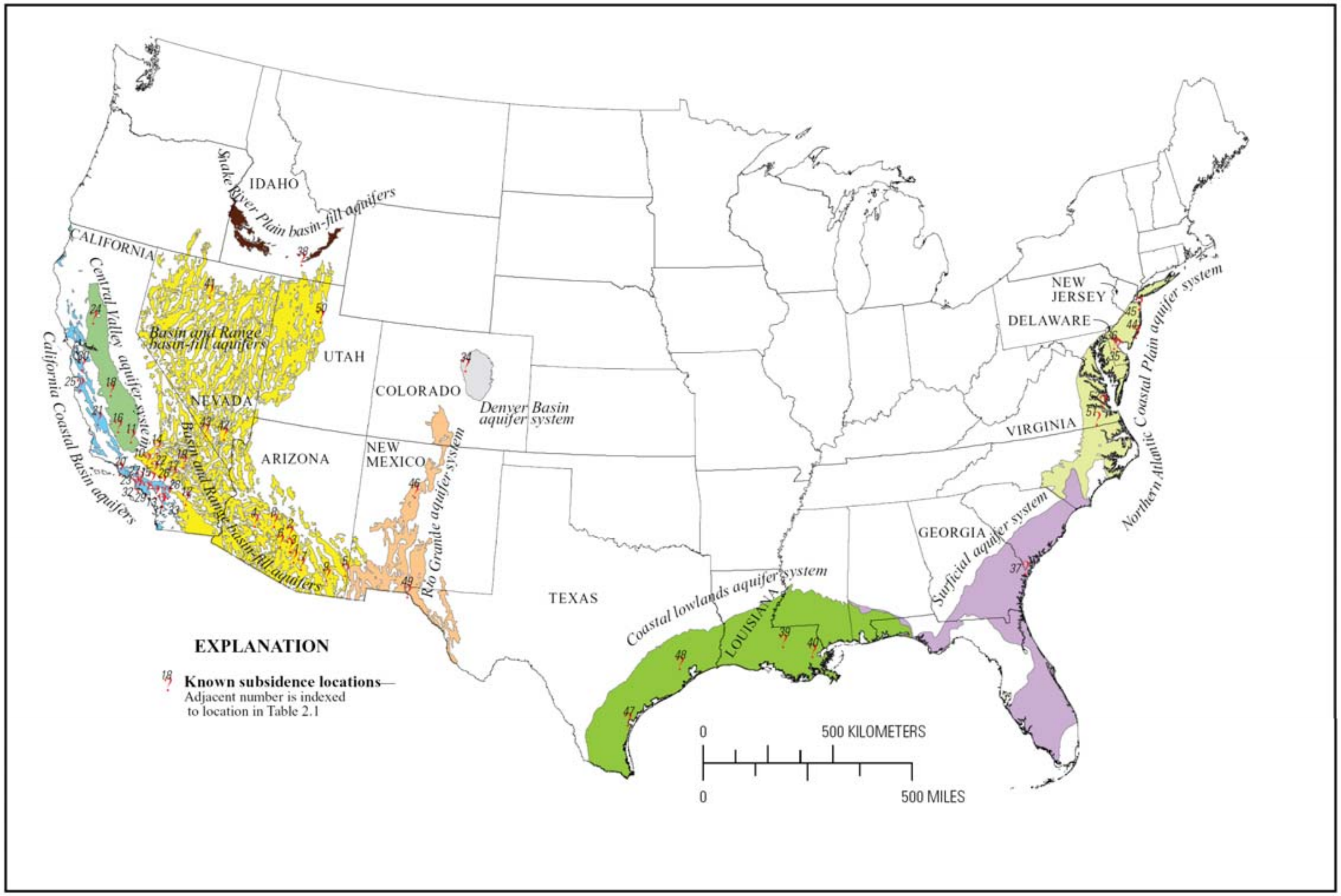


Figure 3.3 Selected, known areas of permanent land subsidence owing principally or secondarily to ground-water or oil and gas extractions in the 48 conterminous United States and associated aquifer systems (modified from Galloway et al., 1999)

Sinkholes

A sinkhole is a closed topographic depression in a karst or pseudokarst area, commonly with a circular or elliptical pattern. The size of a sinkhole generally is measured in meters or tens of meters, rarely in hundreds of meters; and it is commonly funnel-shaped and associated with subsurface drainage (Figure 3.4). Sudden and unexpected collapse of the land surface into subsurface cavities is arguably the most hazardous type of subsidence.

Salt and gypsum underlie about 35-40 percent of the contiguous United States (Figure 3.5) and are, respectively, almost 7,500 and 150 times more soluble in water than limestone. Bedded or domal salt deposits underlie 25 states and constitute about one-half of the area underlain by salt and gypsum. The typically localized collapse features form naturally in relatively soluble evaporite (salt and gypsum) deposits and carbonate (limestone and dolomite) rocks. Human activities often facilitate the formation of sinkholes in these susceptible materials and trigger their collapse, as well as the collapse of preexisting subsurface cavities. Such catastrophic subsidence is commonly triggered by ground-water-level declines caused by pumping or by purposeful or inadvertent diversion of surface runoff enhancing ground-water flow through soluble rocks or into subsurface voids that are partially filled with rock rubble in a matrix of erodible soil material.

The high solubilities of salt and gypsum permit cavities to form over periods ranging from days to years, whereas cavity formation in carbonate rocks is a relatively slow process that generally occurs over centuries to millennia. The slow dissolution of carbonate rocks favors the stability and persistence of the distinctively weathered landforms known as karst.

Underground Mining

In terms of land area affected, underground mining accounts for about 20 percent of the total land subsidence in the United States, and most of this fraction is associated with underground mining for coal (National Research Council, 1991). Subsidence attributed to underground coal mining, generally classified as pit subsidence or sag/trough subsidence, had affected about one quarter of the area undermined or 2 million acres in the United States by the 1970s with the eventual area undermined projected to increase five-fold in the future (HRB Singer, Inc., 1977; Johnson and Miller, 1979).

Most of the mining and subsidence has taken place in the eastern half (east of 100°W longitude) of the United States, mostly in Pennsylvania, Illinois and West Virginia (Gray and Bruhn, 1984). In the western half of the United States in the 1980s underground coal mines occupied about 0.28 percent of the 15-percent portion of western land area underlain by coal deposits, and subsidence has occurred locally above many of these mined areas (Dunrud, 1984).

The subsidence is time-dependent with vertical (generally largest) and horizontal components of movement, depending on the type and extent of mining. Early underground mining was less efficient than more recent underground mining. Subsidence over early mines can occur tens to hundreds of years after mining has ceased, whereas subsidence over more recent mines where virtually total extraction is practiced tends to occur contemporaneously with mining. Subsidence over underground coal workings develops as a gradual downwarping of the overburden into mine voids and is generally unrelated to subsurface water conditions. Mine voids and other subsurface voids can be discovered or located using micro gravimetric surveys. Ground-penetrating radar surveys may be of value in cases where the voids are within a few meters of the ground surface.

Abandoned tunnels and underground mines for metallic ores, limestone, gypsum and salt contribute a small percentage of the subsidence attributed to underground mining. These mined areas are subject to downwarping of the overburden, but limestone, salt and gypsum whether mined or not are susceptible to extensive dissolution by water, frequently leading to sinkholes and in some cases catastrophic collapses (Galloway et al., 1999).

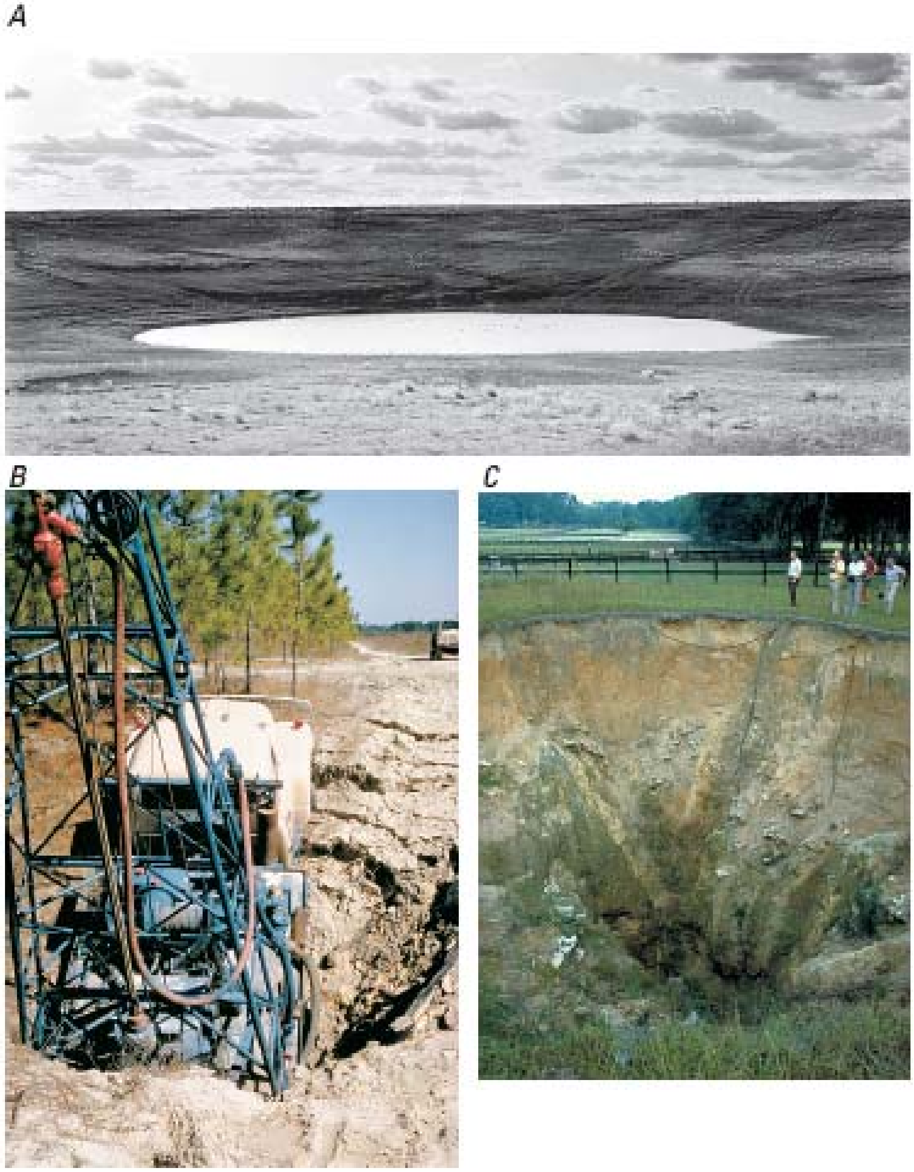


Figure 3.4 Examples of sinkhole formations (A) The Meade sink overlying gypsum and salt beds in western Kansas (Photo from Kansas Geological Survey), (B) Drilling induced sinkhole in carbonates near Tampa, Florida (Photo by Tom Scott), (C) Cover-collapse sinkhole in mantled carbonate karst near Ocala, Florida (Photo by Tom Scott)

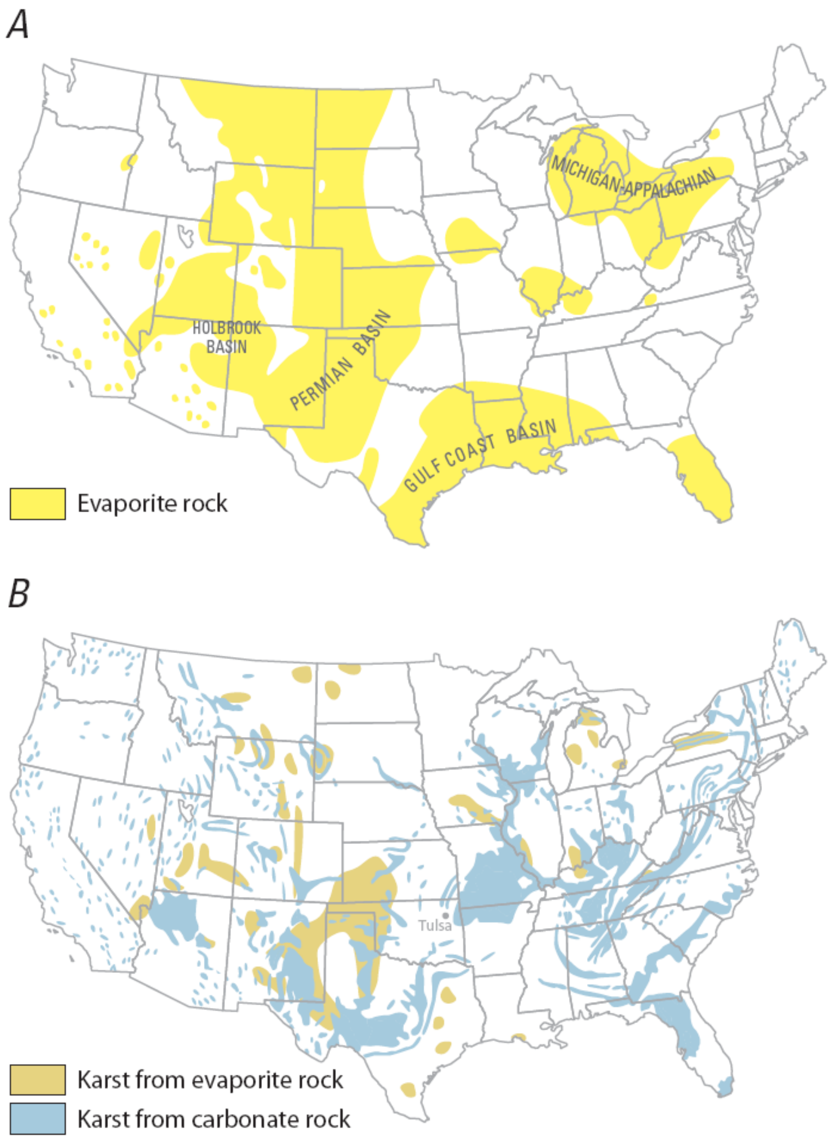


Figure 3.5 Areas prone sinkhole formation in the United States (A) Salt and gypsum (evaporite) deposits (from Martinez et al., 1998), (B) Karst from evaporite and carbonate rocks in the 48 conterminous United States (from Davies and LeGrand, 1972)

Hydrocompaction

Hydrocompaction typically occurs in alluvial-fan sediments above the highest prehistoric water table and in areas where sparse rainfall and ephemeral runoff had never penetrated below the zone subject to summer desiccation by evaporation and transpiration. Under these circumstances the initial high porosity of the sediments (often including numerous bubble cavities and desiccation cracks) is preserved in the deposits. These deposits can support loads due to their high dry strength but collapse when wetted.

Aeolian soils or loess deposits within arid or semi-arid terrain also frequently have low in place densities as a result of cementation or negative pore-suction particle bonding. Similarly, there are numerous cases in which fine-grained soils have been placed as fills of as much as 30 m thickness under dry conditions and without effective watering during compaction, producing settlement when wetted.

Most of the potential hydrocompaction latent in dry, low-density sediments occurs rapidly as the sediments are thoroughly wetted. Thus the progression of a hydrocompaction event is controlled largely by the rate at which the wetting front of percolating water can move downward through the sediments. A site underlain by a thick sequence of poorly permeable sediments may continue to subside for months or years as the slowly descending wetting front weakens progressively deeper deposits. If the surface-water source is seasonal or intermittent, the progression of subsidence is further delayed.

Thawing Permafrost

Thawing of permafrost is one of the key issues identified as potential consequences of climate variability and change for Alaska (Parson et al., 2001) and other Arctic regions. Permafrost underlies about 85 percent of Alaska and varies widely in depth, continuity, and ice content (Figure 3.6). About 50 percent of Canada's land surface lies in the permafrost region, either in the continuous zone where permafrost extends to great depths or in the discontinuous zone where the permafrost is thinner and there are areas of unfrozen ground (Canadian Geotechnical Society, 2006). Thawing permafrost creates thermokarst terrain which is typified by uneven surface topography that includes pits, troughs, mounds, and depressions. Depressions, troughs, and pits which fill with water enhance further thawing of underlying permafrost.

Particular care often is required in the design and construction of pipelines or other infrastructure to prevent changes to the natural site cover and drainage conditions which increase the rate of thermal degradation or the depth and extent of the active zone. Stripping of highly organic muskeg or tundra surficial soils, which provide a natural insulation layer, and placement of granular roadbed or trench backfill materials to facilitate access and construction can result in a significant increase in the seasonal active zone, and resulting subsidence or instability. Thawing will speed organic-decomposition reactions, increase ground-water mobility, increase susceptibility to erosion and landslides, and can lead to further subsidence owing to the oxidation of drained, exposed organic material.

Continuous permafrost on the North Slope of Alaska has warmed 2ºC to 4°C since the late 1800s and more rapidly over the past couple of decades (Osterkamp and Romanovsky, 1996; Hinzman et al., 2005). Because temperatures at the upper surface of continuous permafrost are still low, no significant loss of continuous permafrost is projected over the 21st century. The discontinuous permafrost to the south is warmer, and increased warming suggests that much of the discontinuous permafrost south of the Yukon River and on the south side of the Seward Peninsula could be thawing.

Where permafrost has high ice content, typically in about half the area of discontinuous permafrost, thawing can lead to the development of thermokarst terrain with subsidence, observed in some cases to exceed 5 m. Thawing of ice-rich discontinuous permafrost has damaged houses, required costly road replacements and increased maintenance expenditures for pipelines and other infrastructure (Figure 3.7), and increased landscape erosion, slope instabilities and landslides.

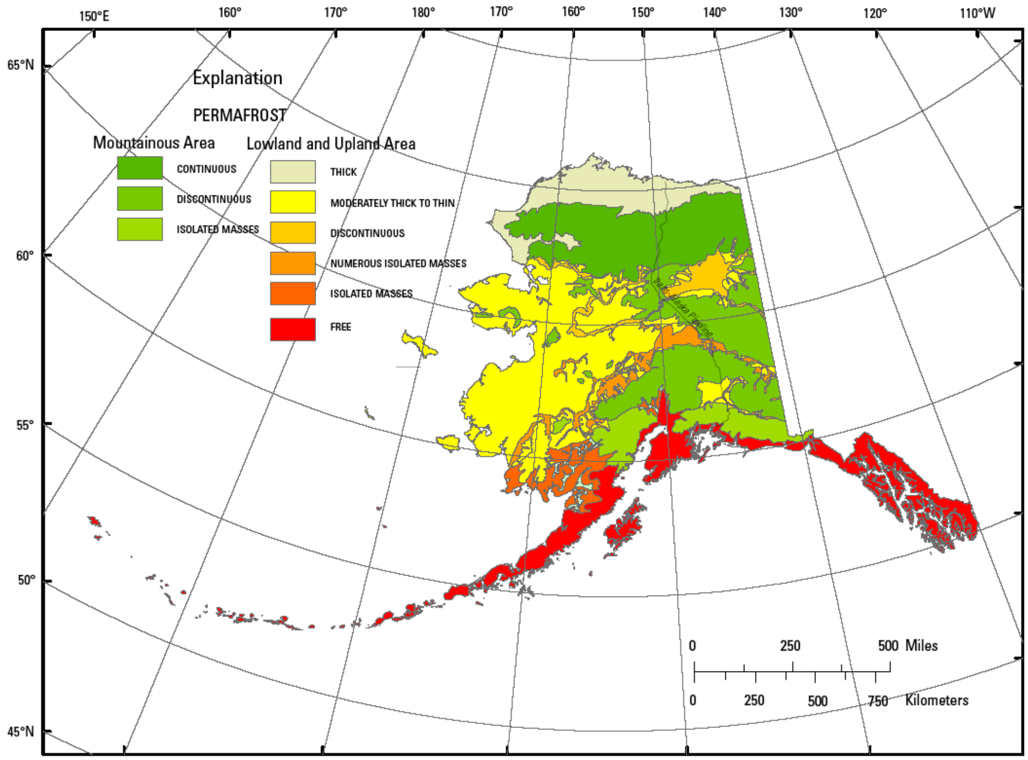


Figure 3.6 Map showing distribution of permafrost in Alaska (modified from U.S. Geological Survey, 1996)

Figure 3.7 Abandoned railroad tracks warped by thermokarst near Valdez, Alaska (During construction of the roadbed the thermal equilibrium of the permafrost was disrupted causing differential thawing)

Consolidation

Rates of natural consolidation generally are low and not likely to pose hazards to pipelines during their operational lifetime. In areas of rapid sedimentary deposition, such as the deltas of large river systems or in topographic basins adjacent to zones of rapid tectonic uplift, long-term average deposition rates may be as large as 1 mm/yr (Ingebritsen et al., 2006, p. 357). As expected, regional subsidence rates attributed to natural consolidation are higher in the Louisiana coastal plain where Holocene sediments derived from the Mississippi flood plain and delta are relatively thick as compared to the Texas coastal plain where the Holocene sediments are relatively thin.

Anthropogenic factors in the developed coastal plains also contribute to subsidence and complicate the determination of natural consolidation rates.

Man-made Consolidation

Construction over soft or compressible soils is often preceded by placing surcharge loads on the surface to accelerate consolidation. The amount and placement duration of the surcharge material varies with the characteristics of the softer underlying material. Likewise, the amount of resulting consolidation can vary greatly with settlements of the near-surface soils of over 1 m not uncommon. This amount of settlement can lead to damage of pipeline coatings or direct damage to pipelines located above the compressible soils.

3.1.2 Identifying Subsidence Susceptible Areas

The soils characteristics and the geology of the surficial and subsurface rocks can be used to help identify subsidence prone areas for each of the types of subsidence listed above. Where this information is unavailable, some field reconnaissance or geologic observations and mapping may be needed. In the absence of other obvious features such as sinkholes (karst terrain), pit or sag/trough subsidence (associated with underground coal mines), other ancillary or anecdotal information that suggests subsidence may be occurring is often useful. Ancillary information is pertinent to regional-scale subsidence processes where the subsidence may be subtle and difficult to detect, therefore much of the emphasis in the following section is on those processes, such as subsidence accompanying the withdrawal of subsurface fluids.

Increased Incidence of Damaged Wells

Protruding well heads and casings are common in agricultural areas and some urban areas where ground water has been extracted from alluvial aquifer systems (Figure 3.8). The land surface and aquifer system are displaced downward relative to the well casing, which is generally anchored at a depth below the compacting layer. Where the frequency of well-casing failures is high, land subsidence is often suspected and is often the cause.

History of Repeated Adjustments to Local Geodetic Controls

Detection of regional land subsidence may be thwarted by the assumption that the bench marks used to establish local geodetic control are stable, that is, located outside the area affected by subsidence. The subsidence may then go undetected until later routine surveys, or until suspicions arise and steps are taken to confirm the current elevations of the affected bench marks.

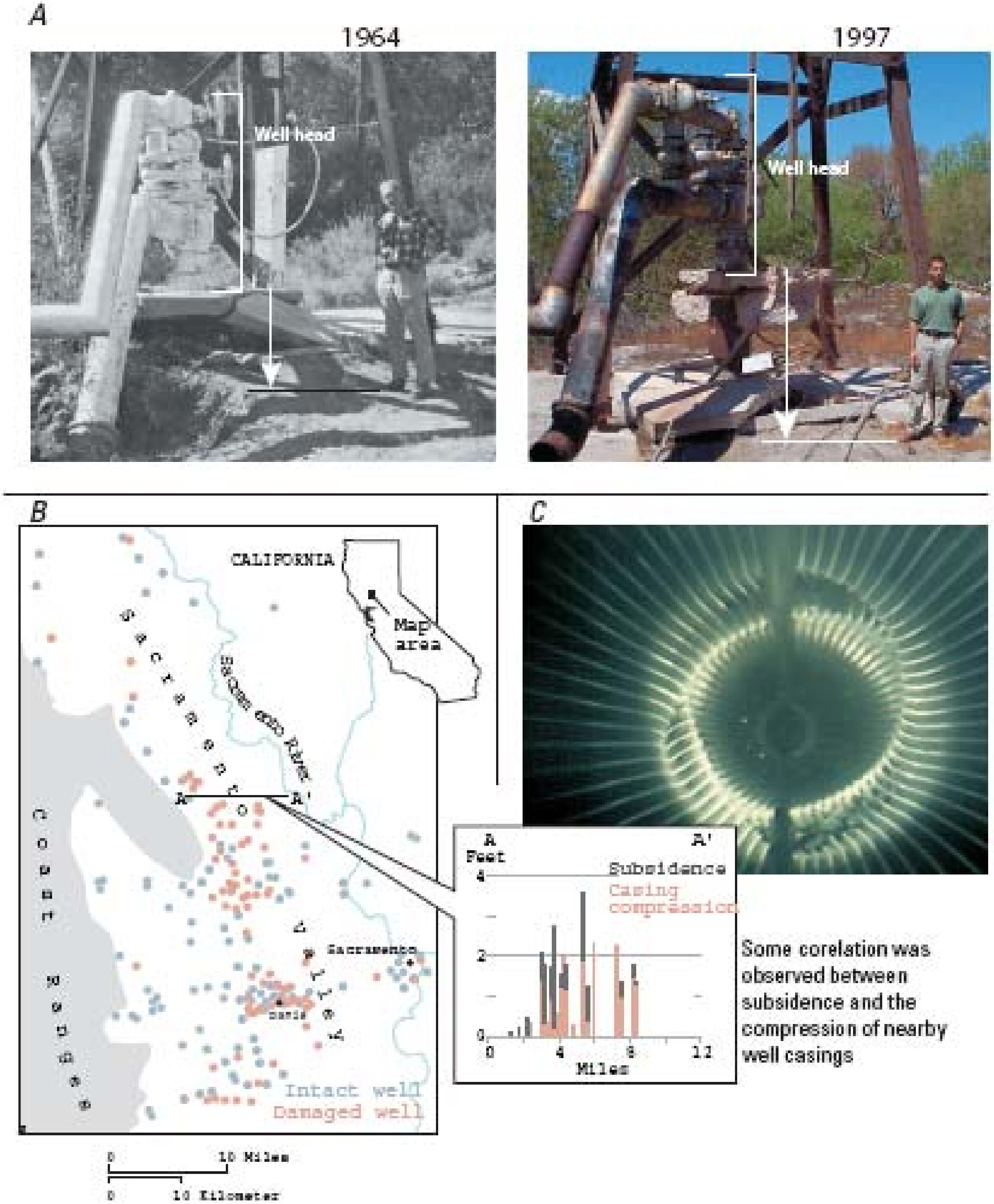


Figure 3.8 Examples showing evidence of subsidence (A) Photographs showing progressively protruding well in Las Vegas, Nevada in 1964 and 1997, (B) Map showing distribution of damaged wells in Sacramento Valley, California and correlation to subsidence along section A-A′ (modified from Borchers et al., 1998), (C) Photograph from borehole camera showing collapsed, spiraled well screen in damaged well

A

B

Figure 3.9 Examples of flooding from subsidence (A) Permanently submerged lands at the San Jacinto Battleground State Historical Park near the shores of Galveston Bay, (B) Homes near Greens Bayou flooded during a storm in June 1989 (Photograph courtesy of Harris-Galveston Coastal Subsidence District)

Increasing Incidence of Local Riverine or Coastal Flooding

Flooding is most severe where land subsides adjacent to water bodies, particularly in coastal regions subject to tidal surges. This causes either permanent submergence or more frequent flooding

Changes of Topographic Gradients

Changes of topographic gradients occur where loss of elevation is not uniform. This may result in stagnation or reversals of streams, aqueducts, storm drainages, or sewer lines; failure, overtopping or reduction in freeboard along reaches of levees, canals, and flood-conveyance structures; and, more generally, cracks or changes in the gradient of linear engineered structures such as pipelines and roadways.

Ground Failures

Two types of ground failures, earth fissure formation and movement on preexisting surface faults, commonly are recognized in association with surface deformation caused by the extraction of subsurface fluids (Figure 3.10). Ground-water pumping in the greater Houston area is attributed as the cause of offsets on more than 86 faults at the land surface with a cumulative length of more than 240 km. These faults, which grow by aseismic creep, have wracked and destroyed many houses, buildings, and buried utilities. Today, surface faults are associated with land subsidence in at least five areas in the United States.

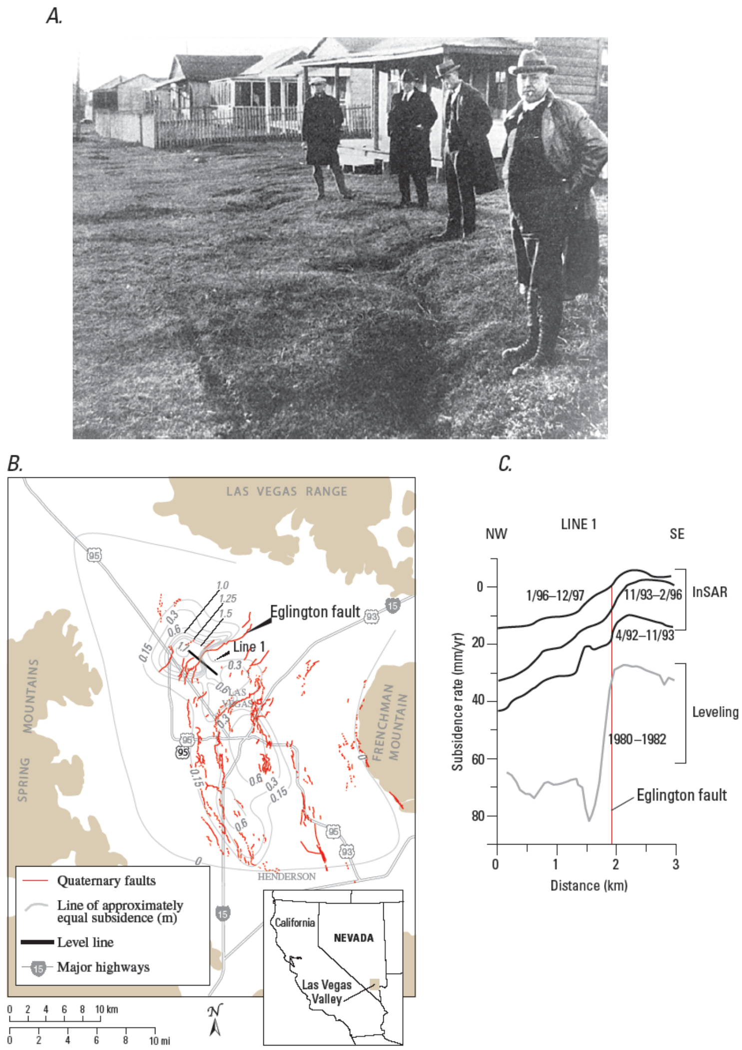


Figure 3.10 Examples of subsidence faulting (A) Photograph (circa 1926) of surface fault near Houston, Texas about one-half mile north of the Goose Creek oil field, (B) Surface faults in Las Vegas Valley, Nevada shown in relation to measured subsidence caused by ground-water pumping, 1963-2000 (modified from Bell et al., 2002), (C) Subsidence rates measured along Line 1 (shown in B.) showing differential subsidence across the Eglington fault (modified from Amelung et al., 1999)

Scarps formed by these faults resemble those caused by tectonism, and the two can be confused, particularly because both typically form along preexisting geologic faults. Scarps range from discrete shear failures to narrow, visually detectable flexures. They grow in height by creep. Observed creep rates in the Houston area range from 4 to 27 mm/yr, which is typical of these faults. The fastest observed creep rate is 60 mm/yr on the Picacho fault in central Arizona (Holzer, 1984). Neither sudden offset nor seismicity is observed on these faults. Detailed monitoring of differential vertical displacements across a few faults reveals that creep rates of individual faults vary with seasonal fluctuations of ground-water level. In addition, long-term changes in creep rate, including its cessation when water-level declines stop, have been reported (Holzer and Gabrysch, 1987).

Earth fissures occur in at least 18 unconsolidated sedimentary basins in 12 areas in the western United States (Holzer and Galloway, 2005). The density of fissures varies greatly between areas. In some places only a few isolated fissures have formed, whereas elsewhere, many fissures occur. Four distinct consequences are posed by fissures: (1) stresses associated with ground displacements associated with their formation, (2) loss of support by ground-surface collapse into deep, steep-walled gullies caused by post-fissure erosion, (3) interception of surface runoff, and (4) erosion of land near the fissure. Although horizontal displacements across fissures during their formation are small, they are sufficient to damage rigid engineered structures. In addition, differential vertical displacements in narrow zones near fissures may affect structures sensitive to small tilts.

The photographs in Figure 3.11 depict earth fissures in south-central Arizona (Carpenter, 1999). The Picacho earth fissure was photographed (Figure 3.11(A) in October 1967 (inset) and June 1989. By 1989 the fissure had developed into a system of multiple parallel cracks with a scarp of as much as 0.6 m of vertical offset. The Central Main Lateral Canal, part of the Central Arizona Project, (also pictured in *A*., upper left of 1989 photo) was damaged by a fissure (Figure 3.11(B) circled) where it crosses the Picacho earth fissure. A natural-gas pipeline undercut by erosional opening of an earth fissure near the Picacho Mountains (Figure 3.11(C)).

3.2 Measurement, Mapping and Monitoring

Subsidence assessments typically address the spatial (magnitude and direction) and temporal changes in the position of land surface, and the process causing the subsidence. Measuring and monitoring subsidence is critical to constrain analyses of the causative mechanisms and forecasts of future subsidence.

Compilations of the available geodetic, geologic, hydrogeologic, mining and cultural information may be available from a variety of sources. Sources of geodetic information and data include local land surveyors, and municipal, state, such as state transportation departments, and federal agencies such as the National Geodetic Survey (United States). Sources of geologic, hydrogeologic and mining information pertinent to subsidence hazards may include local well drillers, hydrologic and engineering consulting firms, state agencies responsible for natural, geologic and water resources, and regulating mining activities, and federal agencies such as the Office of Surface Mining (responsible for regulating the surface impacts of underground mining in the United States), and the U.S. Geologic Survey. For abandoned mines, maps and other information on subsurface conditions frequently are unavailable but some useful information may be available from state divisions of oil, gas, and mining. Historical conditions may be evaluated using available aerial photography and satellite remote sensing data.

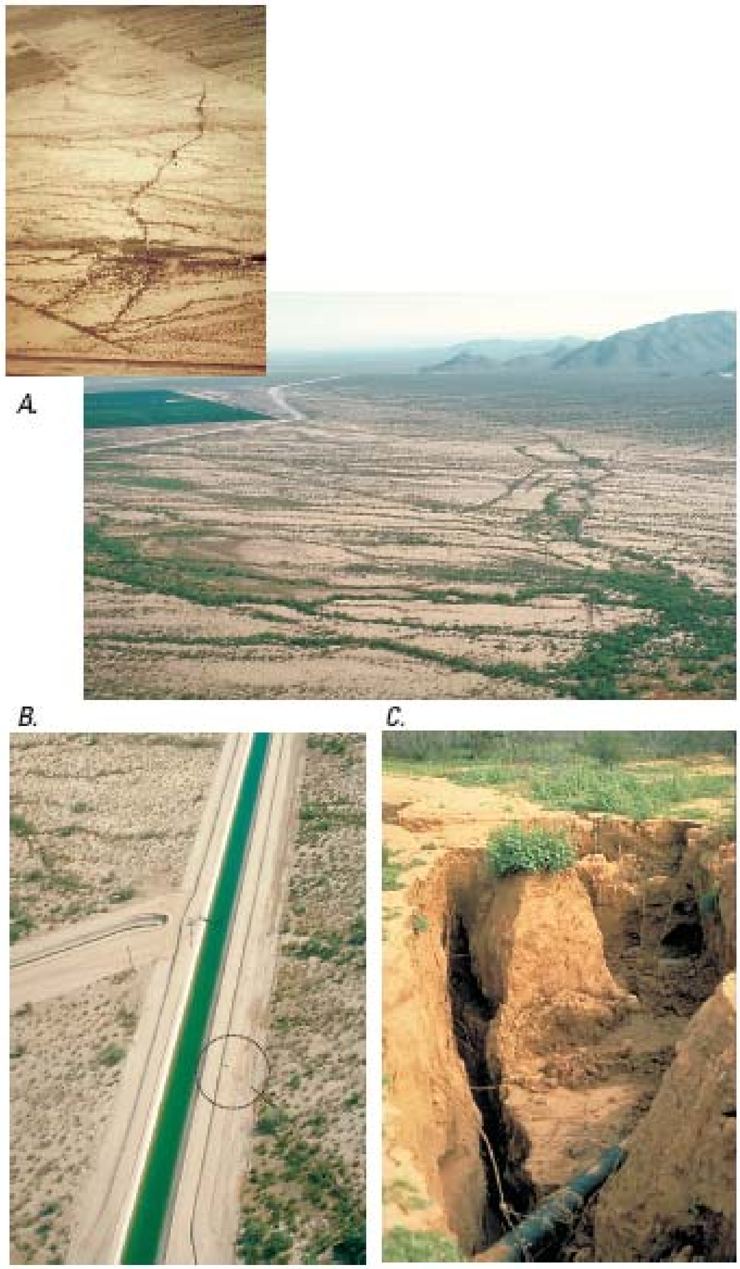


Figure 3.11   Earth fissures in south-central Arizona (modified from Carpenter, 1999)

Various methods used for measuring and mapping spatial gradients and temporal rates of regional and local subsidence and horizontal ground motion are described in greater detail in Baum et al. (2008). The methods generally measure relative changes in the position of the land surface, using a geodetic reference mark so that any movement can be attributed to deep-seated ground movement and not to surficial effects such as thaw settlement. Access to a stable reference frame is essential for the measurements needed to map land subsidence. In many areas where subsidence has been recognized, and other areas where subsidence has not yet been well documented, accurate assessment has been hindered or delayed by the lack of a sufficiently stable vertical benchmark.

3.3 Analysis and Simulation

Analytical models of the particular subsidence process, calibrated or verified using the available data, often are used to assess present and potential future hazards. The analysis of subsidence-prone areas generally involves evaluation of surface and subsurface geologic material properties and the potential physical and chemical processes causing land subsidence. The analyses typically include one or more of the following: laboratory and field measurements, and analytical and numerical modeling of subsidence processes.

3.3.1 Fluid Withdrawal

Numerous numerical models have been developed and applied to simulate subsidence owing to aquifer-system compaction caused by ground-water extractions. Fewer models have been specifically developed to simulate subsidence attributed to other mechanisms. Some general purpose hydraulic-mechanical-thermal models have been used to simulate some of the various other processes.

Because sinkholes result from a combination of many factors, forecasting their spatial and temporal occurrence is difficult. However, if relations between sinkholes and factors associated with their occurrence can be determined, it is possible to assess geologic hazards associated with preexisting sinkholes in karst terrains and risks of new sinkhole formation.

3.3.2 Mining

Surface subsidence due to underground coal mining is generally classified as pit subsidence or sag/trough subsidence. Pit subsidence is a roughly circular hole in the ground with essentially vertical to belled-outward sidewalls (Figure 3.12(A)). The diameter of subsidence pits ranges from about 1 m to 12 m and generally occurs over shallow mines (depths less than about 50 m) with incompetent bedrock overburden. Sag subsidence is a rectangular depression with gently sloping sides (Figure 3.12(B)) and is typically developed over room-and-pillar or longwall-extraction mines at greater depths (20 m to 100 m or more) and with more competent overburden. Trough subsidence is similar in surface geometry to sag subsidence.

Pit and sag subsidence occurs more or less randomly and unexpectedly when pillars of coal or potash collapse under overburden loading, during but potentially years or hundreds of years after an underground mine has been abandoned. Pit subsidence may involve collapse of only a few pillars, while sag subsidence may involve progressive failure of many pillars. Trough subsidence typically occurs in conjunction with longwall mining. The longwall mining technique involves use of moveable hydraulic roof supports, which make it possible to excavate blocks of coal on the order of 300 m wide and about 1,500 m to3,000 m long. Hydraulic roof supports are advanced behind the excavation so the mine roof and overlying rock fracture and collapse into the void behind the supports. Caving and fracturing propagate up through the overlying rock mass and bulking occurs until the collapsed rock supports the overlying strata (Figure 3.12(A)).

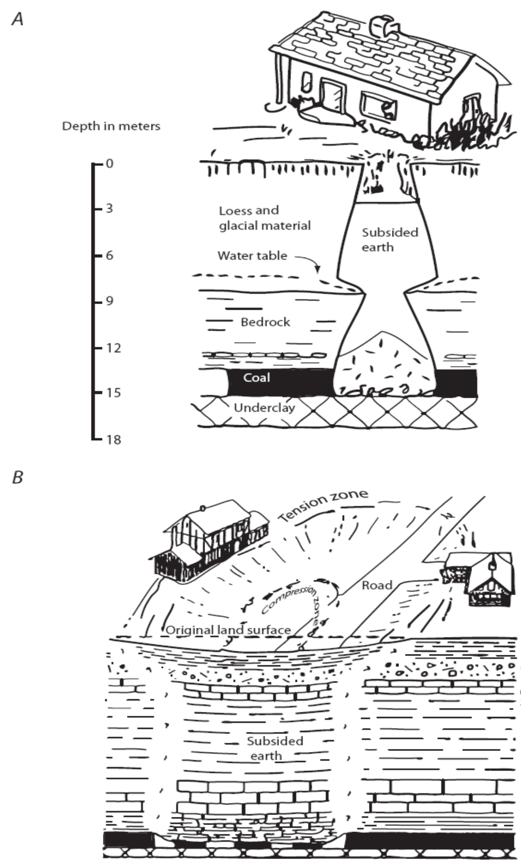


Figure 3.12 Typical types of subsidence associated with underground coal mining, (A) Pit subsidence, (B) Sag subsidence (modified from Bauer and Hunt, 1982)

3.3.3 Hydrocompaction

The susceptibility to subsidence hazards owing to hydrocompaction is difficult to identify in detail and is rarely considered in pipeline siting or design. General geologic settings can be identified where deposits susceptible to hydrocompaction are reasonably likely to be present. The low-density soils necessary for hydrocompaction to occur can be detected in boreholes using geophysical density logs, nuclear density gages or by collecting undisturbed samples and testing for relative density. Hydrocompaction susceptibility largely has been determined by combining soils maps with laboratory data on soil properties to develop maps of "collapse probabilities". Experience in British Columbia indicates that soils or fills having in place densities less than 90 percent of standard Proctor maximum dry density are susceptible to sudden collapse or settlement (Butler, written comunication, 2007). Extensive studies of loess or loess-like soils over a period of many years in China have developed criteria to differentiate the susceptibility of various deposits in differing geographic or climatic regions to sudden collapse or subsidence as a result of wetting (Lin and Liang, 1982).

3.3.4 Thawing Permafrost

During the past two decades a number of permafrost numerical models have been developed to evaluate spatial and temporal changes in permafrost related to global climate change. Despite the importance of permafrost in the climate-change sciences, modeling of permafrost has remained highly diverse and uncertain regarding appropriate methods, their accuracy, and their applicability to different scales and climatic conditions (Shiklomanov et al., 2004). Typical input parameters of a permafrost model include skin temperature at the upper boundary of snow or vegetation cover, the thickness of snow and vegetation covers, and physical properties of soils. At the lower boundary of the domain, the geothermal heat flux generally is prescribed. The principal observational parameters usually are permafrost temperature and the thickness of the active layer, or frost zone, which is the top layer of soil that seasonally freezes and thaws.

3.4 Pipelines and Subsidence Hazards

Subsidence related to fluid withdrawal, longwall mining, and thawing permafrost are hazards that, once recognized, are certain to occur and need to be factored into pipeline design and operational considerations. In contrast, ground displacements related to landslides, sinkholes, and hydrocompaction are random phenomenon that can not be predicted with great certainty.

In addition to the certainty of some subsidence hazards, there are typically few options available to reduce the severity of the hazard. With the exception of thawing permafrost related to pipeline construction and thermal operating conditions of a pipeline, the underlying processes are the result of actions by others such as regional water users, oil field development companies, or by changes in global climate.

4.0 PIPELINE RESPONSE TO GROUND DISPLACEMENT

Experience in the oil and gas industry with respect to the analytical evaluation of buried pipeline response to large permanent ground displacement dates back to the mid-1970s. This experience includes simple evaluation methodologies as well as more sophisticated finite element approaches. A brief summary of the history and the experience of pipelines undergoing large ground deformations are presented in 2004 PRCI seismic design guidelines (Honegger and Nyman, 2004).

All simple evaluation methodologies that rely on calculations that can be carried out by hand or using a spreadsheet are practically limited to consideration of initially straight pipelines, single components of lateral offset (either vertical or horizontal), constant soil-pipeline interaction parameters, and negligible pipe bending stiffness. Nonlinear finite element techniques are the only means available to analyze all but the most simple of problems. Finite element approaches provide a means to rapidly investigate the effects of changes in backfill characteristics, pipeline material, wall thickness, and pipeline alignment. There are no restrictions on the analysis software that can be used as long as it is capable of capturing the non-linear effects of non-linear soil springs, user-defined stress-strain curves for the pipe material, and large changes in pipeline geometry. The analysis software should include a pipe element in its element library with the capability to model internal pressure and provide output at various circumferential locations.

The recommended analytical approach for the analysis of pipeline response to permanent ground displacement from landslide and subsidence hazards is generally very similar to the PRCI seismic guidelines (Honegger and Nyman, 2004), although the details of the methodology in these guidelines incorporate the findings of recent research into pipe-soil interaction.

4.1 Overview of Approach

The recommended approach for performing an analysis of pipeline response to permanent ground displacement requires representing the condition of continuous pipeline embedment by discrete axial, vertical, and horizontal soil springs as illustrated in Figure 4.1.

Movement of the surrounding soil with respect to the buried pipeline may force the pipeline to move with the soil or result in differential movement between the pipe and the soil. A key characteristic of soil loading is that it increases only to the point at which gross failure of the soil occurs. Capturing this characteristic requires a non-linear representation of the soil springs.

A comprehensive review of relationships developed to represent soil-pipe interaction is contained in C-CORE (2003). The maximum soil spring forces and associated relative displacements necessary to develop these forces are computed using the equations given in Appendix A. The expressions for maximum soil spring force are based upon laboratory and field experimental investigations on pipeline response, as well as general geotechnical approaches for related structures such as piles, embedded anchor plates, and strip footings.

Several of the equations have been derived to fit published curves to facilitate use in spreadsheets or other computer-based applications.

The recommendations for defining soil springs for analysis of pipeline response include several modifications to the recommendations in Honegger and Nyman (2004) that are worth noting:

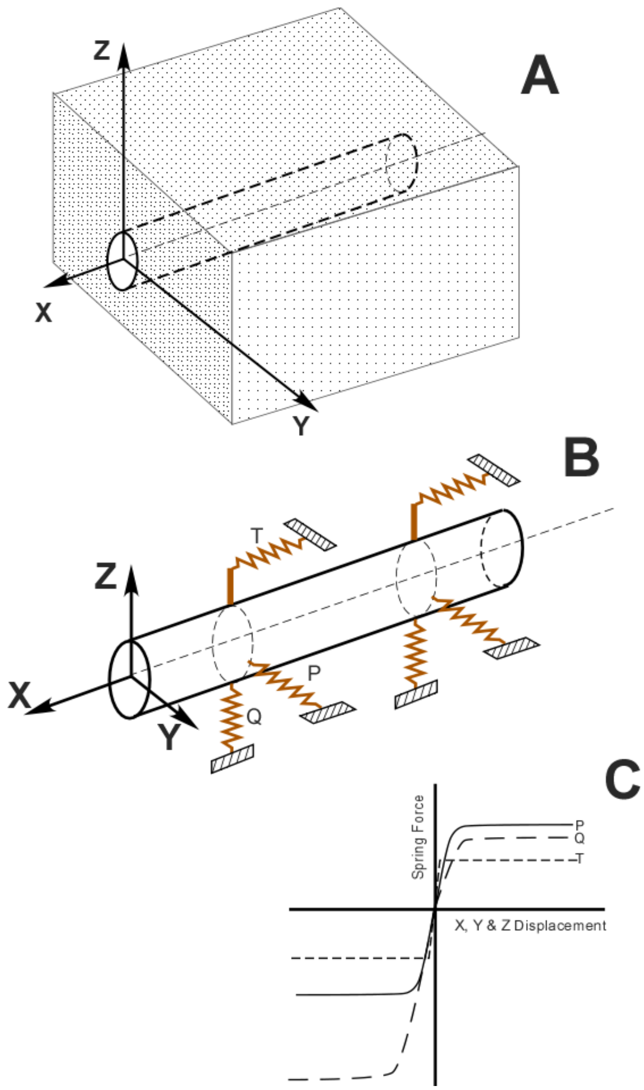


Figure 4.1 Spring analog for analyzing pipeline-soil interaction

- As slope movements and subsidence typically involve smaller displacements than those typical of seismic hazards, appropriate characterization of soil stiffness is generally more important. Soil spring stiffness characteristics are provided based upon hyperbolic formulations presented in ASCE (1984) and other more recent publications.

- Recent experimental investigations (Honegger et al., 2006, O'Rourke et al., 2008) have confirmed that there is no increased lateral soil resistance in moist sand as reported by Turner (2004). Lateral soil spring definitions are based upon dry sand as recommended in O'Rourke et al. (2008).
- Lateral soil resistance for drained and undrained loading in clays are based upon recommendations developed by C-CORE (2003, 2008).
- Recommendations for alternate axial soil resistance relationships are provided for sand considering conditions in which the sand may be dilative.
- Recommendations are provided for accounting for various trench effects based upon work by C-CORE (2003) and Honegger et al. (2006).

4.2 Recommendations for Modeling

Detailed recommendations for implementing finite element analyses of pipeline response to ground displacement are provided in Appendix A. This section provides a summary of some of the more important factors to be considered.

4.2.1 Extent of Pipeline Model

The length of pipeline modeled outside of the zone of applied ground movement is dependent upon the specific pipeline alignment. The length of pipeline needs to be sufficient to assure that small elastic deformations outside the area of high pipeline strains do not significantly change the magnitude of the highest computed longitudinal strains and that the pipeline model adequately captures the anchoring effects of the soil outside the zone of ground movement. Extending the pipeline model a considerable distance outside the zone of ground movement typically does not significantly lengthen the solution time for the analysis as the pipeline response in this region is mostly elastic. An estimate of the extent of the model for which the pipe can be considered anchored to the soil, L_{anchor}, can be computed using the following formula:

$$L_{anchor} = \frac{\pi D t \sigma_y}{T_u} \tag{4.1}$$

where

D	=	pipe diameter
t	=	pipe wall thickness
σ_y	=	pipe yield strength
T_u	=	maximum axial soil force acting on pipe (see Appendix A)

The length L_{anchor} is the distance necessary for the axial soil force, T_u, to generate axial yield in the pipeline. In cases where a particular analysis program is impacted by an extended pipeline model, a shorter length can be used if analyses are performed to confirm that longer models do not appreciably change the maximum computed strains.

4.2.2 Pipe Element Definition

The elements representing the pipeline should be capable of accounting for the effects of internal pressure because the hoop stress from internal pressure reduces the longitudinal compression stress at which the material will yield. The pipe element length in regions where the pipe strain is expected to exceed the yield strain (typically at abrupt transitions in ground displacement or locations with abrupt changes in soil restraint such as elbows) generally should not exceed one pipe diameter. The one-diameter limitation on the length of pipe elements in areas of high strain is related to the gage length typical of full-scale pipeline tests typically used to define pipeline strain acceptance criteria (see section 4.3). Multiple straight or curved pipeline elements can be used to model elbows and bends in the pipeline alignment. The maximum length of pipeline elements used to model elbows should not represent more than a 15° angular change (e.g., at least 6 elements to model a 90° elbow). It is preferable to use multiple curved pipe elements at bends and elbows if they are available. In most cases, the differences in results between using curved pipe elements versus many straight pipe elements are not significant.

Stress intensification and flexibility factors are based upon elastic pipeline response and are not applicable to analysis of nonlinear pipe behavior. The use of multiple straight pipe elements to represent bends and elbows introduces localization of strains that may be overly conservative considering the greater flexibility typical of elbows. This conservatism in the treatment of elbows is warranted because of the lack of sufficient tests to determine applicable strain limits for elbows. However, tests on Grade B elbows have demonstrated that they have the ability to withstand severe deformation while maintaining internal pressure integrity (Yoshizaki et al., 1998, 2001).

It is not uncommon for pipelines to be constructed using sections of elbows cut to provide angle changes other than standard 90° or 45°. The wall thickness of elbow fittings is typically not uniform around the circumference and will vary for different manufacturers. Modeling of elbow fittings often requires special attention because of the potential for a significant mismatch in wall thickness between the pipe and the elbow section. This mismatch in wall thickness can lead to stress risers and reduce the strain capacity of the welded connection.

4.2.3 Pipe Stress-Strain Definition

Nonlinear material representation in analysis software is typically based upon a definition of a uniaxial engineering stress-strain curve that is converted to a true stress-strain curve within the software application. Users typically have several options to choose from regarding the modeling of plasticity. Bilinear or multilinear isotropic hardening rules based upon a von Mises yield criterion are adequate for monotonic, non-cyclic loading. Alternate approaches to modeling plastic behavior can also be adopted provided they have a sound basis in engineering mechanics.

4.2.4 Soil Spring Definition

The definition of soil springs assumes that the spring forces always act in the axial, horizontal, and vertical directions relative to the pipeline. In most analyses, the soil springs are defined in a global coordinate system with the result that the direction of the soil spring forces will not maintain an axial, horizontal, and vertical orientation relative to the pipeline if the pipeline undergoes large rotations. However, the error introduced by this misalignment is acceptable considering other assumptions and uncertainties inherent in the analysis.

If the situation being analyzed is one in which bi-directional behavior of the soil springs is possible, consideration needs to be given to the unloading characteristics of the soil springs. Relative pipe-soil displacements are permanent in the sense that the soil does not "spring back". Therefore, soil spring forces should quickly drop to zero or unload along a path parallel to the initial soil spring stiffness as indicated in Figure 4.2 (top).

An example in which it is necessary to properly account for soil spring unloading is the case of a pipeline experiencing upheaval buckling as a result of ground displacement parallel to the pipeline. At the point at which upheaval buckling occurs, compressive forces in the pipeline will be relieved through vertical displacement, resulting in a reversal of the direction of relative axial movement between the pipeline and the soil and causing the soil to limit the ability of the pipeline to feed into the buckled region. As a consequence of this, the compressive axial force in the pipeline that can be relieved through upheaval buckling is limited to the force corresponding to the elastic compressive strain in the pipeline induced by ground displacement. Of course, additional ground displacements that occur following upheaval buckling will increase the vertical displacement at the buckled section of the pipeline.

The situation can be more complicated for transverse loading conditions, as illustrated in Figure 4.2 (bottom), because soil cohesion can result in a permanent gap forming between the pipeline and surrounding soil. Upon reversal in the direction of pipeline displacement, the pipeline experiences minimal resistance until the gap is closed. However, the impact of this behavior is most significant for cyclic loading situations such as seasonal frost-heave, thaw-settlement, high thermal differential, etc.

There is considerable uncertainty regarding the relationships used to compute soil spring properties. Most of this uncertainty is related to estimates of the soil strength parameters. The uncertainty in estimating soil strength parameters for pipeline analyses in cohesive soils is further complicated by the fact that pipelines are typically located above the water table and within the desiccation zone of the soil. The strength of partially saturated desiccated soils is not well defined in soil mechanics practice.

Current analysis techniques assume the equivalent soil springs act independently, a common assumption for analytical representation of pile foundations and similar buried structures. This assumption can introduce errors related to the potential for different soil restraint for oblique soil displacement relative to the pipe (e.g., a combination of horizontal and vertical displacement) and the dependency on soil spring force at a particular point along the pipeline on adjacent relative pipe-soil displacement. The error associated with the assumption of independent soil springs is generally small relative to the overall uncertainty typically associated with defining the soil strength properties in the equivalent spring formulations.

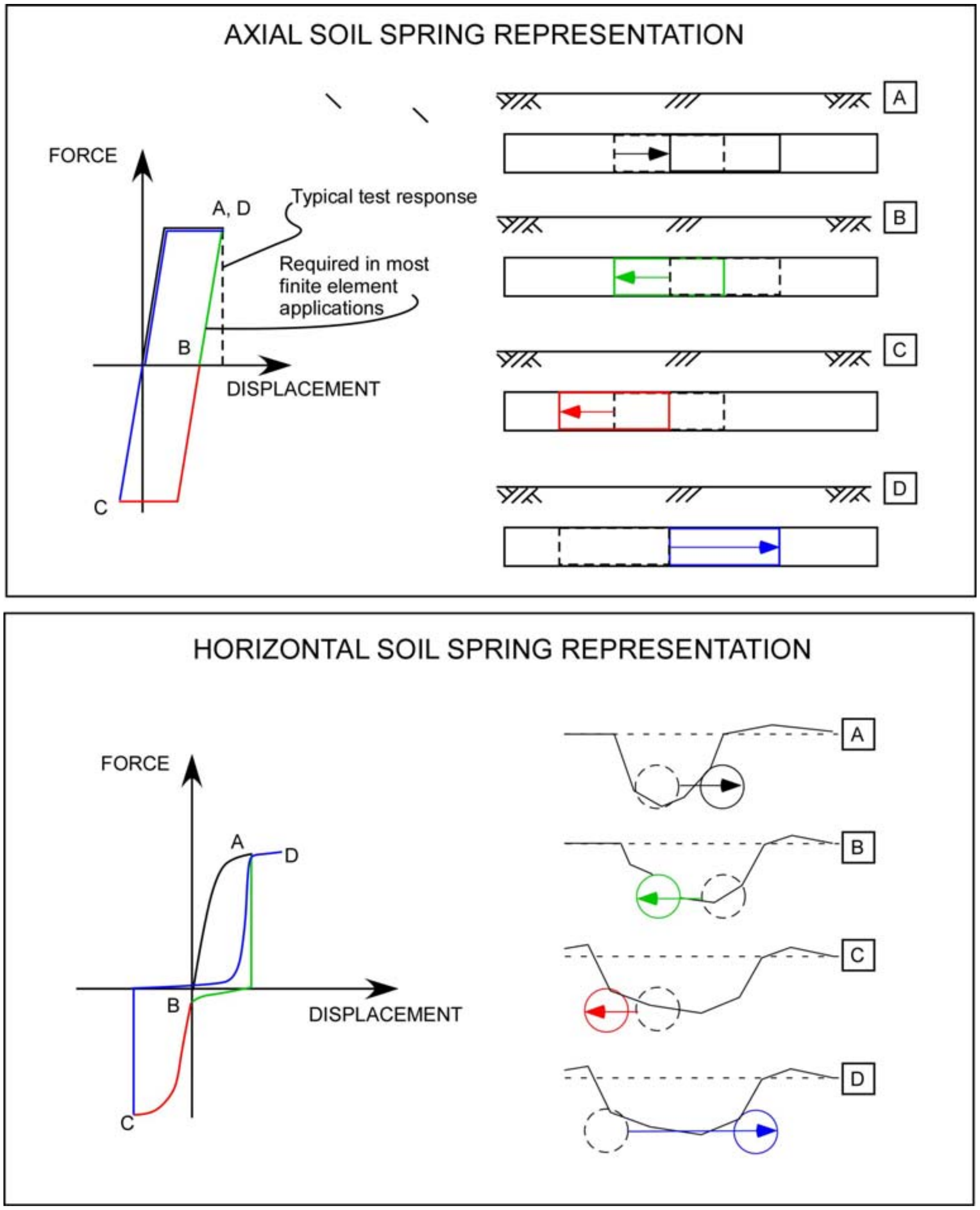


Figure 4.2 Schematic representation of loading and unloading for axial soil springs (top) and horizontal soil springs (bottom)

It is recommended that the definition of soil springs does not explicitly capture the reduction in peak soil force transmitted to a pipeline that can develop at large relative pipe-soil displacements. The reasons for not accounting for this force reduction are primarily related to current approaches for defining pipeline strain acceptance criteria (see section

4.3) but are also related to the limited information available to quantify the load reduction that might actually occur.

The analysis approach using beam-like pipe elements in the analysis of pipe response is only compatible with soil loading representations that consider a reduction in soil force at large relative displacements if the pipeline longitudinal compression strains are below the level corresponding to the onset of pipe wall wrinkling. If these pipeline strain limits are not satisfied, the modeling of the pipe should be modified to capture the localized straining that occurs in the vicinity of a buckle in the pipe wall. This will typically require modeling a section of the pipeline where the wrinkling strain limit is exceeded with shell elements or solid elements that can capture the wrinkling behavior and subsequent reduction in pipeline load capacity (force or moment).

Approaches to quantify the variation of soil load with displacement are either based upon soil mechanics (i.e., two-dimensional or three dimensional computer analyses) or experimental test data. Analytical approaches require several assumptions on soil constitutive properties which may be determined in a research setting but are rarely known in actual practice. Experimental investigations are typically performed on uniform soils (e.g., well characterized sand or reconstituted clay mixtures) under carefully controlled conditions and often at reduced scale, making direct extrapolation to actual field conditions subject to considerable uncertainty.

4.2.5 Soil Properties

The key soil properties used to define soil springs are the unit weight of the soil and either internal friction angle and cohesion for drained assumptions or friction angle (sand) or undrained shear strength (clay) for undrained assumptions. These parameters are best provided by a geotechnical specialist. In most cases, considerable judgment is necessary to account for the variety of factors affecting soil properties such as partial saturation, desiccation, consolidation effects, rate effects, permeability, etc.

Soil properties representative of the backfill should be used to compute axial soil spring forces. Other soil spring forces should generally be based upon the native soil properties. Backfill soil properties are appropriate for computing horizontal and upward vertical soil spring forces only when it can be demonstrated that the extent of pipeline movement relative to the surrounding backfill soil is not influenced by the soils outside the pipe trench.

4.2.6 Representation of Applied Ground Movement

Ground movements representative of the displacement patterns and amplitudes for landslides are applied to the base of the soil spring elements. The ground deformations should be specified based upon estimates of relative ground movement at the depth of the pipeline. Ideally, the expected pattern of ground displacement should be determined on the basis of geotechnical field investigations of similar slide movements, but such may not be feasible in all cases considering the spatial extent of pipelines and the potential for numerous hazard areas.

It is always conservative to assume that the ground displacement occurs abruptly as abrupt ground displacements have been observed at the heads and margins of some landslides. In some locations, it may be possible to infer future ground displacement patterns from past ground failures. For locations with little experience to aid in the determination of ground deformation pattern, an abrupt offset cannot be ruled out without the assistance of a specialist in geology or geotechnical engineering.

An alternate approach for expressing landslide ground displacement is to assume a pattern defined by a cosine function raised to the power of *n* as given in Equation (4.1).

$$y(x) = \delta\left[1 - \left(\cos\frac{\pi x}{W_s}\right)^n\right] \qquad (4.1)$$

where:

$y(x)$ = displacement as a function of location, x, within the zone of deformation

x = distance across the zone of deformation

δ = amount of displacement

W_s = width of slide zone

n = even integer

Example cosine functions given by Equation (4.1) are graphed in Figure 4.3 for various powers of *n*. Selecting a ground displacement pattern other than an abrupt offset should always be done in consultation with the geotechnical specialist responsible for defining the landslide hazard.

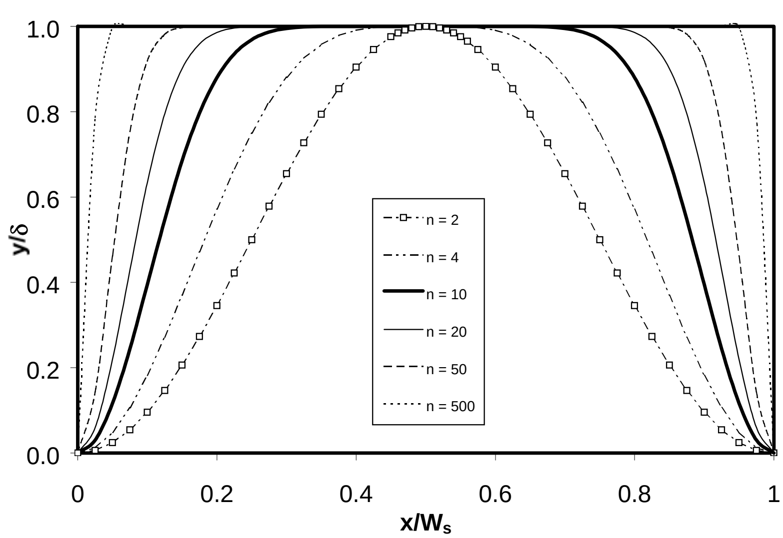


Figure 4.3 Cosine function for representing variation in ground displacement patterns

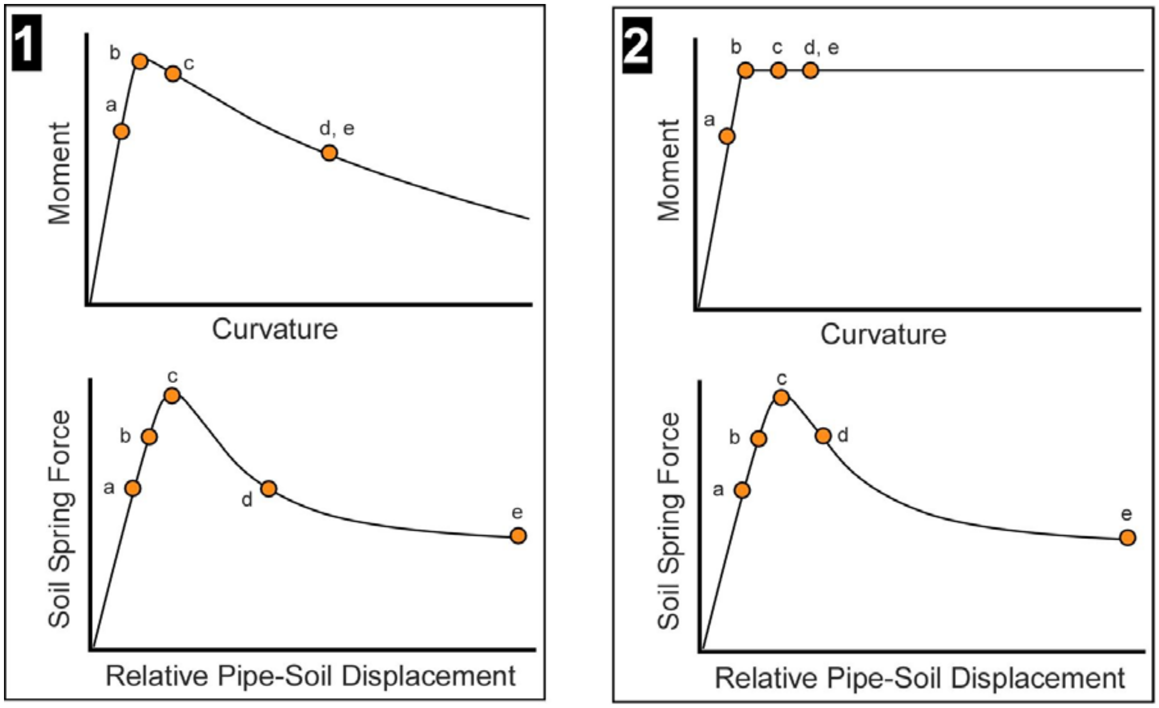
4.3 Relationship Between Pipeline Analysis Method and Strain Acceptance Criteria

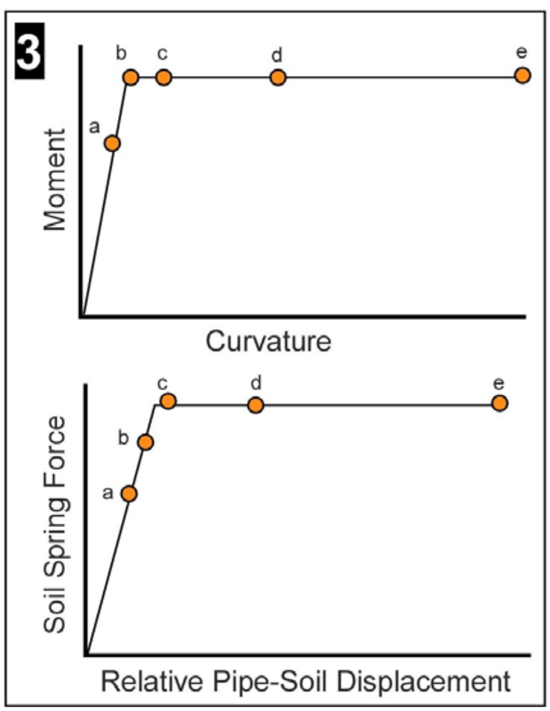
For pipeline load conditions produced by an imposed displacement, relatively large strains can be accepted provided the pipeline is in good condition and the girth welds are capable of developing gross-section yielding of the pipe. For situations where the pipeline is subjected to high longitudinal compression strains from a combination of axial and bending loads, there may be a potential for the development of local buckling of the pipe wall.

Pipeline test results demonstrate an abrupt softening in the moment versus curvature behavior of pipelines with low internal pressure once strains exceed the levels associated with maximum moment capacity, typically associated with the onset of local wrinkling or buckling of the pipe wall (Yoosef-Ghodsi et al., 1994, Mohareb et al. 1994, Zimmerman et al., 1995). Localized straining that occurs once pipe wall buckling initiates is not captured by pipe elements, leading to an underestimate of local and global post-buckling strains in the region of the buckle.

Not capturing the reduction in moment capacity that accompanies formation of a local buckle in the pipe wall is not a significant shortcoming provided the representation of the soil loading does not account for the reduction in maximum soil load at large relative pipe-soil displacement. The use of a constant maximum soil load forces continued deformation of the pipeline once yielding is reached in the pipeline. Localization of bending strains is not a concern if the strain acceptance criteria for the pipeline are based upon the deformation of a section of pipe that encompasses the wrinkle (typically one or two pipe diameters) and not the local strains in the wrinkle.

A highly idealized comparison of the effects of different treatment of soil spring definitions and methods of pipeline modeling is illustrated in Figure 4.4. As noted in Figure 4.4, the primary consequences of adopting a soil-spring force displacement relationship with post-peak reduction without a requirement that local pipe wall wrinkling is captured in the analysis is the possibility that pipeline strains will be underestimated. The recommended approach, (constant peak soil spring force and pipeline modeled with pipe elements) will generally overestimate the pipeline strains.





Box 1: An analysis that represents actual soil force behavior and captures shell response of the pipeline limits the maximum pipe curvature to a level in which force equilibrium is achieved between the pipe and the soil.

Box 2: Using pipe elements in an analysis captures the maximum pipe moment capacity but not the loss of moment capacity past the peak associated with pipe wall wrinkling. If the soil spring force is modelled as reducing from the peak at point c, the pipe curvature only increases to the point where force equilibrium is achieved at point d. This can lead to a substantial underestimate of pipe strain.

Box 3: The recommended approach where no post-peak reduction is modelled for the soil spring forces and moment capacity provides a much better approximation of the pipe curvature that occurs between relative pipe-soil displacement points c and d. The pipe strain at relative displacement point e is overpredicted. However, this is generally insignificant since there is typically minimal additional ground displacement capacity that can be achieved at pipe strains greater than that associated with curvature point d.

Figure 4.4 Impact of approaches to modeling soil spring force and pipeline moment-curvature response

4.4 Comment on Three-Dimensional Continuum Analyses

Many researchers have investigated the applicability of three-dimensional continuum models to compute pipeline response to large ground displacements (see discussion in C-CORE, 2003). While continuum analyses hold the promise of eliminating many of the simplistic representations of soil-pipe interaction using pipe elements and soil springs, several significant obstacles remain to be overcome, many of which are shared by simple soil spring analogs, before continuum analysis methods can be considered superior to pipe element and soil spring representations for routine engineering applications:

- Large Model Size: Providing sufficient numbers of elements to capture local pipe wall buckling and the response at soil discontinuities (e.g., interfaces between bedding, backfill, and in-situ soil), combined with the fact that significant soil-pipe interaction effects may extend over hundreds of meters of pipeline, results in extremely large analytical models that may take days to run using normally available computational resources. Furthermore, the complexity of the model makes it much more difficult to extract results of interest and incorporate relatively minor changes (e.g., change of soil cover or pipe diameter).
- Limited Relative Pipe Displacement: The most widely-used approaches to assess the viability of continuum analyses to assess pipeline response rely on finite-element or finite-difference techniques in which the pipe and soil are represented by a discrete mesh of solid elements. Without an efficient means to reformulate the model mesh, the amount of pipe displacement that can be represented in the analysis is limited to approximately half a pipe diameter because of severe mesh distortion.
- Inability to Capture Non-Continuum Behavior: Full-scale and centrifuge tests have demonstrated soil response during relative pipe displacement exhibits both flow and fracture behavior with slip planes developing within the soil mass as well as at the pipe-soil interface. Continuum models have an advantage over simplified methods in that they can improve the ability to identify when such behavior is important. However, in actual engineering applications, continuum models do not represent a significant improvement over simplified methods.
- Requirements for More Detailed Soil Property Descriptors: Representing soil as a continuum requires much more information on the strength and deformation characteristics of the soil than is necessary for the simple soil spring analogy. This information typically requires a suite of laboratory tests of both the native soil and backfill, and may require calibration of soil constitutive relationships. Considering the limited information that is typically available for a soil spring characterization, the reliability of continuum analyses are likely to be no better than the simple soil spring model.
- Limited Validation: Data available for validation of continuum models are largely the same obtained from tests used for developing simple soil spring definitions: soil load on a short section of pipe versus pipe displacement with observations of surface and subsurface soil deformation patterns. Both the simple soil spring models and a

continuum models select material parameters that match this basic test data. One of the greatest potential benefits of continuum models is the ability to investigate more complex pipeline-soil interaction relationships. Past validation tests do not typically include information on the state of stress within the soil and at the pipe-soil interface, information that is critical for the validation of more sophisticated capabilities of a continuum model. Thus, the validity of results from continuum models is likely to be no greater that what is obtained from simple soil spring models.

Experimental investigations that have simulated the response of a continuous steel pipeline to ground displacement are extremely limited. Of particular note are tests performed at Cornell University (Yoshizaki et al., 2001) and proprietary tests performed as part of a multi-industry funded study (C-CORE, 1998). As of 2008, a research program at Cornell University is investigating the response of a polyethylene pipe to simulated fault displacement. This research is likely to provide much needed information for validating continuum analysis methods.

4.5 Representation of Applied Ground Movement

Ground movements representative of the displacement patterns and amplitudes for lateral spreads or landslides are applied to the base of the soil spring elements. Ideally, the expected pattern of ground displacement should be determined on the basis of geotechnical field investigations, but such may not be feasible in all cases considering the spatial extent of pipelines and the potential for numerous hazard areas. Practically, current empirical or analytical methods for quantifying ground movement hazards are not capable of quantifying the behavior at boundaries. The most appropriate basis for estimating more gradual transitions in ground movement is historical evidence.

It is always conservative to assume that the ground displacement occurs abruptly. Abrupt ground displacements have been observed at surface faults, as well as the head of landslides and lateral spreads. For locations with little experience to aid in the determination of ground deformation pattern, an abrupt offset cannot be ruled out without the assistance of a specialist in geology or geotechnical engineering. If the basis for a more gradual transition is judgment, the sensitivity analyses should include an abrupt transition in ground deformation pattern.

Additional options for representing ground displacement patterns associated with slide-type ground displacements are provided in Appendix A.

4.6 Sensitivity Analyses

The selection of material properties, soil strengths, and ground movement patterns is an inherently uncertain process. Generally, additional analyses should be performed to provide information on the sensitivity of the computed strain levels to changes in input parameters when the level of conservatism associated with the input parameters is not well understood. This information can be used to better define the “best estimate” of pipe response and provide information that can be used to assess the level of confidence in the expected pipeline performance. The range of variation to be used in these sensitivity analyses generally can be estimated in conjunction with the selection of baseline parameters and available information from geotechnical investigations of soil properties and ground deformations that have been performed. Unless other information is available to determine the amount of variation, the following are suggestions for examining the sensitivity of the results obtained by analysis:

- Upper-bound estimates of pipe material strength (as opposed to specified minimum values)
- Variation in soil strength to capture the reasonable range of upper-bound and lower-bound ranges
- Increased applied ground displacement (this can normally be done at the time of the analysis)
- Modifications in the ground displacement transitions at boundaries with zones of potential ground movement

The above variations typically result in a range of acceptable ground displacements or a range of strains for a particular ground displacement value. If a balanced range of parameters is selected, the "best estimate" of pipe response can typically be approximated as being centered within the range of pipeline responses determined from the variation in input parameters.

5.0 PIPELINE DESIGN MITIGATION ALTERNATIVES

Other than avoidance, there are three basic options, used alone or in combination, to improve the response of pipelines to ground displacements from geohazards: increase pipe strength, modify pipeline alignment, and reduce soil loads. Selection of a particular approach is dependent upon considerations that vary with pipeline location, land-use constraints, expected failure mode, potential for collateral damage, risk acceptance philosophy, and mitigation costs. Many of the options may have limited applicability because of topographic constraints, constructability considerations, ability to procure necessary right-of-way access, the need to avoid existing subsurface structures and utilities, or the compaction requirements associated with various types of land use. Urban environments are particularly restrictive with respect to the feasibility of mitigation options to improve pipeline response.

5.1 Increase Pipeline Capacity to Resist Ground Displacement

In some cases, modifications to proposed or existing pipeline configurations can greatly improve performance. Such modifications include increasing the pipe strength by increasing wall thickness or increasing the strength and toughness of the pipe material, and replacing sharp bends and elbows with induction bends or gradual pipeline field bends.

Increasing the pipe wall thickness increases the allowable longitudinal compression strain and increases the bending and axial strength of the pipeline relative to the soil. If maintaining pipeline pressure integrity is the primary design goal, increasing the pipeline wall thickness will also generally increase the allowable longitudinal pipeline strain as the increased wall thickness reduces the influence of postulated weld defects on weld tensile strain capacity.

Increasing the yield strength of the pipe steel can be an effective means to increase pipeline resistance for steels. However, the benefits of using pipeline steels with yield strengths above API Grade X70 may be offset by the need for special measures to assure the girth welds overmatch the pipeline yield and ultimate strengths.

Replacing short-radius or long-radius elbows with induction bends or field bends and locating expansion loops near boundaries of expected ground displacement (Figure 5.1) may increase the ability of the pipeline to distribute axial soil loads over a longer length of pipeline and can decrease the severity of strains developed as a result of local bending at the elbow. The degree to which induction bends or field bends can be an effective mitigation measure primarily depends upon the bend angle and the ground displacement pattern the pipeline is exposed to. Significant improvement in pipeline performance is more likely when the angle change is greater than about 15º or the elbow is located near the margins of the zone of ground displacement.

Figure 5.1 Expansion loop installed to improve axial pipe response to ground displacement

5.2 Reducing Soil Loads on the Pipeline

The capacity of a buried pipeline to withstand ground displacements can be improved by minimizing the longitudinal, lateral, and uplift soil resistance to pipe movements. Potential options for implementing changes to modify the soil loading on buried pipelines are summarized below.

5.2.1 Loose Granular Backfill

A practical means for achieving minimum soil restraint is to bury the pipeline in a shallow trench filled with a loose granular backfill. As depicted in the top diagram of Figure 5.2, the trench walls should be sloped at an angle of about 30° to 45° for horizontal ground displacement components and about 60° for vertical ground displacement components. For horizontal ground displacement conditions, the base of the distance between the pipeline and the wall of the trench experiencing movement toward the pipeline should be equal to the expected horizontal ground displacement. This trench geometry will allow the soil to fail within the backfill material rather than in the higher strength, undisturbed soil outside the trench. Lower diagram is for a situation where downward vertical ground displacement will make the pipeline ride out the ground.

For typical pipeline trench conditions, loose granular backfills (sand or gravel) will offer less resistance to pipe movement than compacted cohesive backfill materials (clay or silty clay). A granular material with an angle of internal friction of 35° or less is recommended.

To satisfy an angle of internal friction of 35° or less, the backfill material should be a clean (non-cohesive), relatively uniformly graded granular material with 100 percent of the aggregate less than 25 mm (1 in) in diameter. The material should be obtained from a natural, rounded or subrounded fluvial deposit that is not dominated by grains of feldspar or other minerals that split along cleavage surfaces and remain angular; crushed rock is not acceptable. All sizes of crushed rock material are angular. Angular fragments have higher angles of internal friction and coefficients of friction on other materials, including corrosion protection coatings on pipelines. The backfill should be placed as loose as possible recognizing time-dependent increase in density is unavoidable.

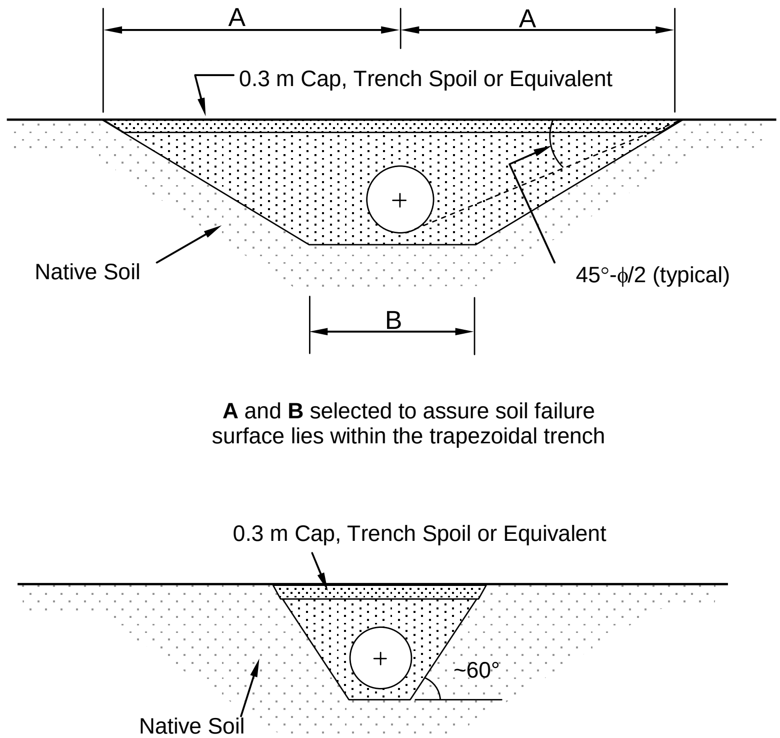


Figure 5.2 Typical configurations used with loose granular backfill for horizontal ground displacement (top) and vertical ground displacement (bottom)

Contractors may choose to excavate vertical-walled trenches for ease of excavation. The assumption of any particular soil condition and the development of spring restraint properties must be consistent with field conditions. In particular, for horizontal relative ground displacement, the soil failure surface generated by pipeline displacement must be contained within the limits of the excavated pipe trench that is backfilled with the selected material.

Similarly, for vertical relative ground displacements, the upward breakout must occur within the designated backfill. If these trench excavation and backfill requirements are not satisfied, soil parameters applicable to the hybrid situation of in situ soils and trench backfill must be included in the development of soil restraints for the pipeline model, and these restraint properties are typically much higher than for loose to moderately dense granular backfill soils.

5.2.2 Low-Friction Coating or Protective Wrapping

Axial soil friction loads can be further reduced over what is achieved by loose backfill by the use of smooth, hard, low-friction coatings. Where suitable backfill is unavailable or maintaining the backfill in a loose condition is impractical, reductions in axial soil friction can be obtained by using two separate layers of geosynthetic wrapping as shown in Figure 5.3.

This type of installation forces axial slip to occur at the interface between the two layers of geotextile fabric and can reduce the interface friction angle used to calculate maximum axial soil spring force to less than 21°, comparable to what can be achieved with loose sand backfill and a smooth, hard pipe coating. The reduction in axial soil friction force obtained from a double geotextile fabric wrapping should be based upon interface friction tests (e.g., tests in general accordance with ASTM D5321) of the candidate geosynthetic fabrics under overburden soil pressures representative of the actual installation conditions. Testing may also be required to determine the long-term impact of aging on interface friction properties.

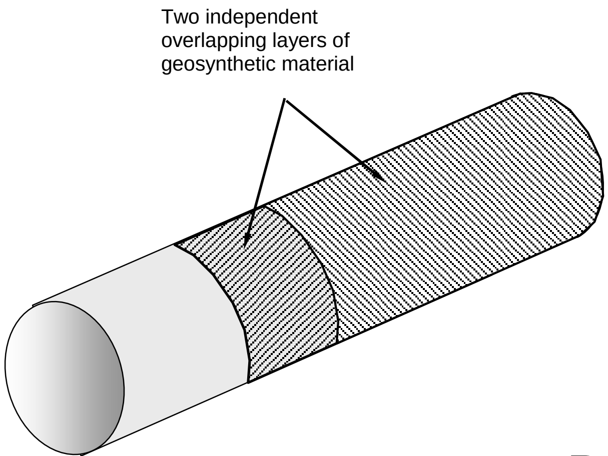
5.2.3 Geosynthetic Lining of Sloped Trench Walls

Additional reduction in horizontal soil load over what can be achieved using loose granular backfill in conjunction with sloped trench walls (see 5.2.1) is possible by lining the walls of the trapezoidal trench with two layers of geosynthetic fabric. The two layers of geosynthetic fabric create a low-friction failure surface in lieu of the logarithmic spiral failure surface that would be developed in the backfill material. The load necessary to overcome the friction between the two layers of geosynthetic fabric is much less than that required to develop a shear failure in the backfill soil.

5.2.4 Replacing Soil with Lightweight Materials

Geofoam or other lightweight materials offer a means to reduce axial, lateral, and upward vertical soil loads. Geofoam is a rigid cellular plastic foam of either expanded polystyrene (XPS) or extruded polystyrene EPS). Geofoam has been used extensively in northern Europe for subgrade insulation in regions susceptible to thaw settlement. Another usage of geofoam in Europe and the U.S. is as low-density fill for construction over weak or compressible soils. One common application is to use geofoam as fill for bridge approaches and abutments. Geofoam varies in weight from about 160 N/m^3 (1 lb/ft^3) to 470 N/m^3 (3 lb/ft^3). Compressive strength of XPS is generally less than EPS although the compressive strength of both increases with density. The typical range of compressive strengths is 140 kPa to 240 kPa (20 psi to 35 psi) for EPS and 200 kPa to 500 kPa (30 psi to 75 psi) for XPS.

A



B

C

Figure 5.3 Illustration of dual-layer geosynthetic material application, (A) Conceptual drawing, (B) Examples of field applications, (C) Wrapped pipe ready to be backfilled

Other lightweight materials, such as pumice or expanded shale, may also be considered as a means of reducing loadings on pipelines. Although of higher density, typically 800 to 1000 N/m^3 (50 to 65 lb/ft^3) compared to geofoam, these materials can typically be handled and placed in the same manner as a granular fill. In some areas, natural peat or woodwaste having consistent, generally small particle sizes such as wood chips or bark stripping (hogfuel) materials can be considered for use, as they combine both low density, typically less than 550 N/m^3 (35 lb/ft^3) and high compressibility. However, use of woodwaste or other organic materials such as straw may not be desirable or acceptable adjacent to pipelines, unless they are fully saturated and therefore at low risk of combustion.

Replacing soil above the pipeline with geofoam or other lightweight materials reduces axial friction force by effectively reducing overburden stresses acting normal to the pipeline. Care must be taken to maintain a proper balance between limiting pipeline restraint for ground movement, yet providing sufficient restraint to prevent upheaval buckling of straight pipe and excessive bending stress at pipe bends due to operating load conditions.

Lateral loads on buried pipelines also can be reduced by using geofoam and other lightweight materials to replace much of the backfill soil as shown in Figure 5.4. This load reduction is achieved due to two factors. First, the boundary between soil and geofoam forms a failure surface that is weaker than that corresponding to the failure surface of the in situ soil. Second, the weight of material displaced along the soil-geofoam boundary is much less. The shearing force can be reduced even further by placing loose sand between the geofoam and the native soil or through the use of dual layers of geosynthetic fabric at the interface.

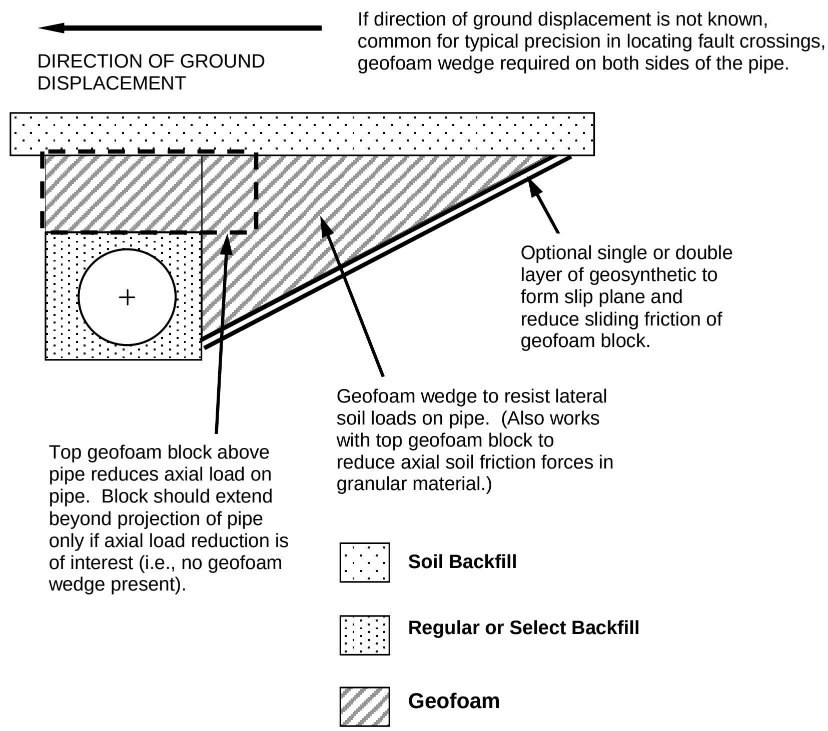


Figure 5.4 Conceptual illustration of the reduction of soil loads using geofoam

The use of geofoam has some advantages in urban settings because the compressive strength of the geofoam is sufficient to handle light traffic loads. A potential significant drawback to geofoam applications includes increased vulnerability to applied external loads, need to prevent contact with gasoline and other organic fluids or vapors, high flammability, and cost.

5.2.5 Use of Controlled-Strength Material

Controlled strength material around the pipeline can be used to limit the lateral loads that can be exerted normal to the pipe wall, allowing the pipe to bend in a more gradual manner to accommodate the imposed ground displacement. Geofoam is one type of material that has been used for this purpose since it can experience large compression strain under near-constant compression load. Cellular concrete is another material that has appropriate crushing characteristics, although the loads to initiate crushing are often much higher than geofoam. Cellular concrete mixture of sand, cement, and water to which a foaming agent or polystyrene beads are added to create small air pockets. The use of crushable material as a mitigation measure is typically only a practical consideration in situations where trapezoidal trench construction is not practical and the in-situ soil strength results in very large lateral soil loads to the pipeline.

The distance from the pipeline where controlled-strength material is needed is largely governed by the amount of compressive irrecoverable strain that can be accommodated before the material begins to exhibit much higher compressive strength. This strain level is commonly referred to as the lock-up strain. The lock-up strain for cellular concrete can vary from 15% to 35%. The lock-up strain for EPS or XPS geofoam can vary from 25% to more than 50%.

5.3 Modifying the Pipeline Alignment

In some cases, relatively minor modifications of the pipeline alignment through and adjacent to areas of potential ground displacement can significantly reduce the strains induced in the pipeline from ground displacement. The goals of pipeline alignment modifications are generally to reduce the length of pipeline exposed to ground displacement, eliminate points where soil-generated loads can concentrate along the pipeline, and maximize the flexibility of the pipeline to accommodate imposed deformations.

5.3.1 Reducing the Length of Exposed Pipeline

Reducing the length of pipeline exposure requires a clear definition of the extent of the region of potential ground displacement and the direction of ground displacement. Optimum pipeline crossing alignments will be those that limit the length of pipeline within the hazard and provide an orientation that results in a combination of axial and horizontal soil loading that poses the minimum strain demand for the pipeline. For example, a change in pipeline alignment to take advantage of the case where the pipeline capacity for ground displacements parallel to the pipeline is much greater than the capacity for ground displacements perpendicular to the pipeline, as illustrated for a hypothetical case in Figure 5.5. The opportunity to implement pipeline alignment modifications is often restricted by the lack of an ability to acquire suitable right-of-way or potential construction difficulties associated with the modified alignment (e.g., access limitations, steep slopes).

5.3.2 Maximizing Unanchored Length

The capacity of a buried pipeline to withstand ground displacement components can be improved by maximizing the distance from the deformation zone (fault rupture, landslide, lateral spread, etc.) to points of virtual anchorage created by the pipeline alignment, typically side bends, overbends, and sagbends. Sharp bends, tees, branch fittings, valves, etc. also will have a tendency to anchor the pipeline against axial movement and should be avoided within or near a zone of ground displacement. Good design practice is to provide a straight segment of pipeline as long as practical through and beyond the ground displacement zone to maximize the length of pipeline available to distribute strain.

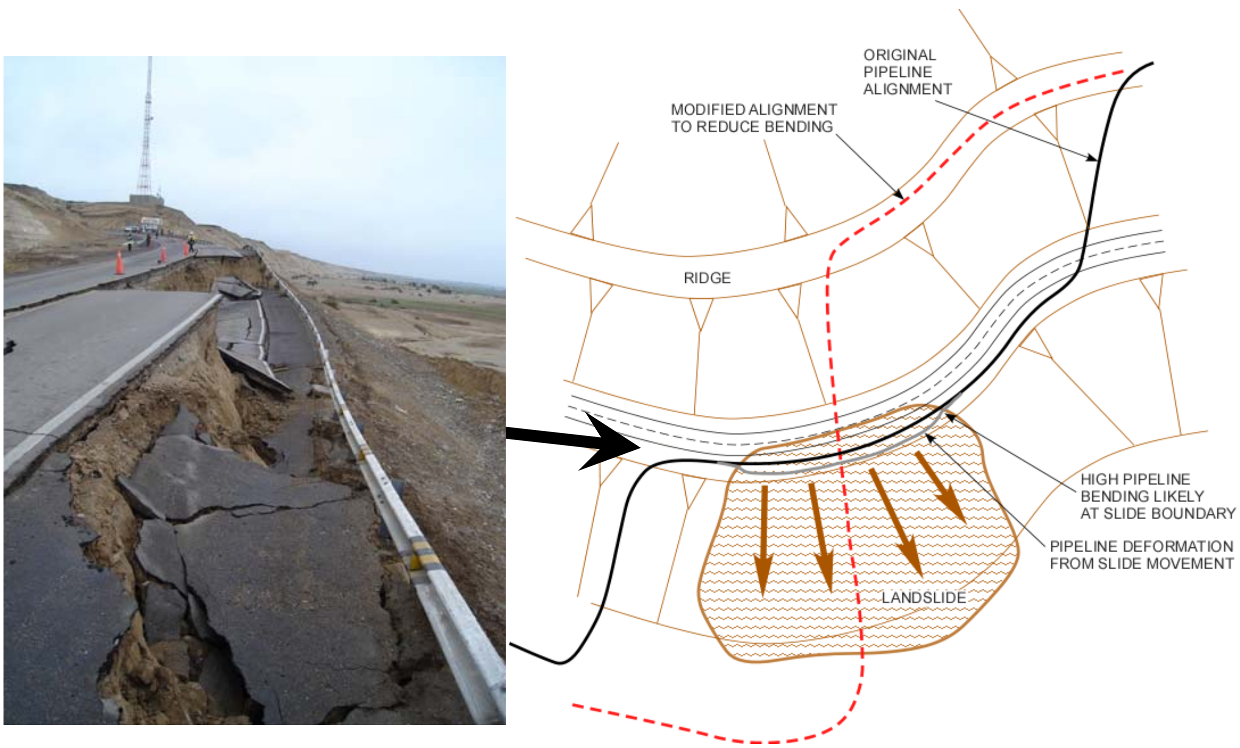


Figure 5.5 Hypothetical example of alignment modification to take advantage of greater pipeline resistance to axial soil displacement

5.3.3 Isolating Pipelines from Ground Displacement

When pipelines must cross landslide hazard zones, avoidance can be achieved by locating the pipeline above or beneath the active slide zone. Bridging a hazard zone may be a practical alternative when the dimensions of the slide are not great. For relatively shallow slides, it may be possible to locate the pipeline beneath the active zone of slide displacement using conventional trenching techniques. When conventional construction is not possible, directionally drilled options may be feasible.

Locating Pipeline Aboveground

Lateral soil loads can be greatly reduced by placing the pipeline on the ground surface or on aboveground supports. Typically this is done by attaching sliding shoes to the pipeline that bear on structural steel members tied to the ground or mounted in an aboveground configuration. Several options for such a modification are shown in Figure 5.6. Teflon, or other low-friction materials, can be incorporated into the construction of the sliding shoes to improve the ability of the pipeline to accommodate ground displacement by sliding laterally. Where soil cover is required to protect the pipeline from third-party damage, an earthen berm can be used in lieu of burial. Locating pipelines above ground is rarely a practical solution outside of controlled access areas or very remote regions. Two types of aboveground supports used on the Trans-Alaska pipeline are shown in Figure 5.7.

Steep and narrow zones of potential slide movement may be bridged with an aboveground crossing. Bridging is most effective if the pipeline can span the hazard with only two supporting elements. In some cases, it may be possible to span the potential hazard zone with no additional pipe support (Figure 5.8), although this practice is not presently very common. If the pipe is unable to span the unstable area, additional supports or an external guyed support system will be required as illustrated in Figure 5.9. If pipe supports are required within the slide zone, they must be founded in competent material below the slide plane and be designed to withstand the soil loads that will develop if sliding occurs (e.g., passive lateral soil load, impact of soil debris above the support).

Install Beneath Unstable Area

In many cases, it is possible to avoid landslide hazards by locating a pipeline below the slide failure plane. For shallow slides, this can be accomplished using conventional trenching methods. Horizontal directional drilling (HDD) techniques can be utilized to place a pipeline below a deep slide plane and has become a more common method of landslide avoidance with broader use of HDD to install pipelines with minimal surface disturbance at river crossings and through environmentally sensitive areas. However, sufficient laydown area is required for pulling the pipe string through the completed HDD boring. Often the topographic conditions in landslide areas limit the available laydown area needed to complete an HDD operation.

Accurate location of the slide plane is critical to this method of avoidance as increasing the depth of pipeline burial greatly increases the potential for serious pipe damage if movement occurs in the soil surrounding the pipeline. This is particularly important for HDD crossings of landslide hazards as the soil depth over the pipeline generally makes access to the pipeline to make subsequent repairs extremely difficult, if not impossible. Soil borings and downhole instrumentation often provide sufficient information to deduce the slide plane location. Consideration should be given to direct observation to confirm these findings by drilling and downhole logging large (1 m or less in diameter) borings for particularly critical pipeline installations.

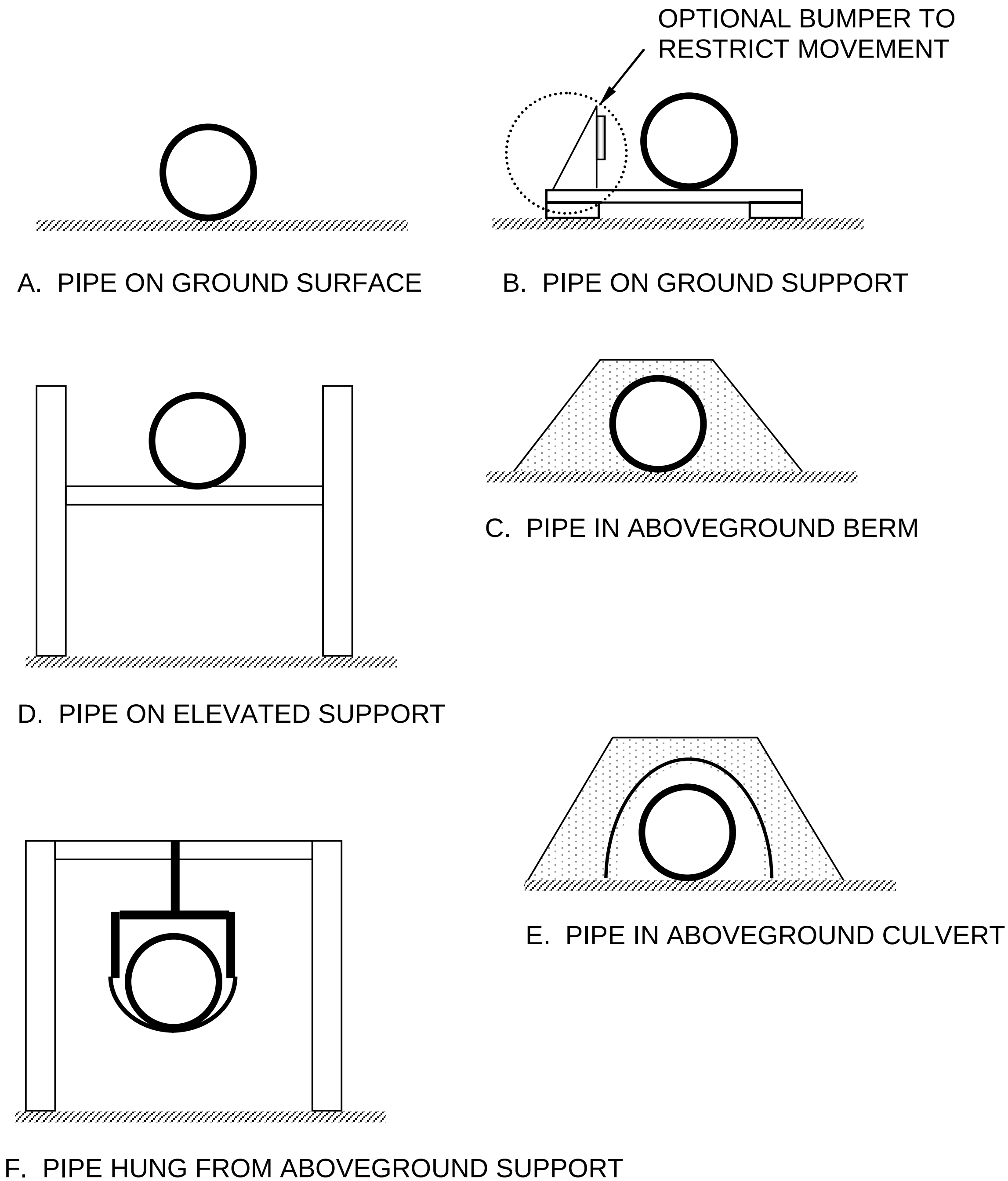
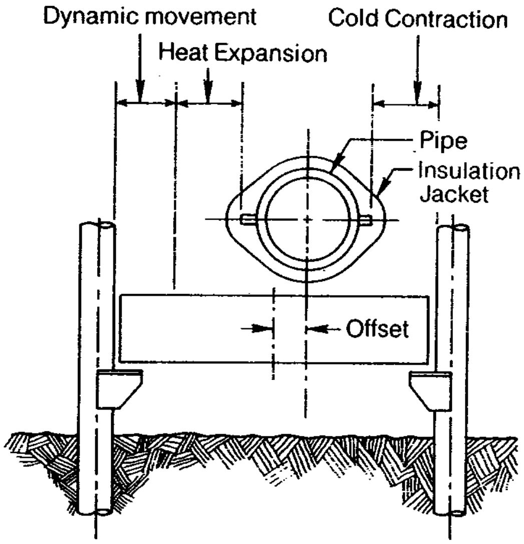


Figure 5.6 Aboveground pipeline support alternatives



Cross section of intermediate aboveground support and photograph of sliding shoe utilized at the support

Typical bent on a concrete grade beam

Figure 5.7 Aboveground supports for Trans-Alaska Pipeline

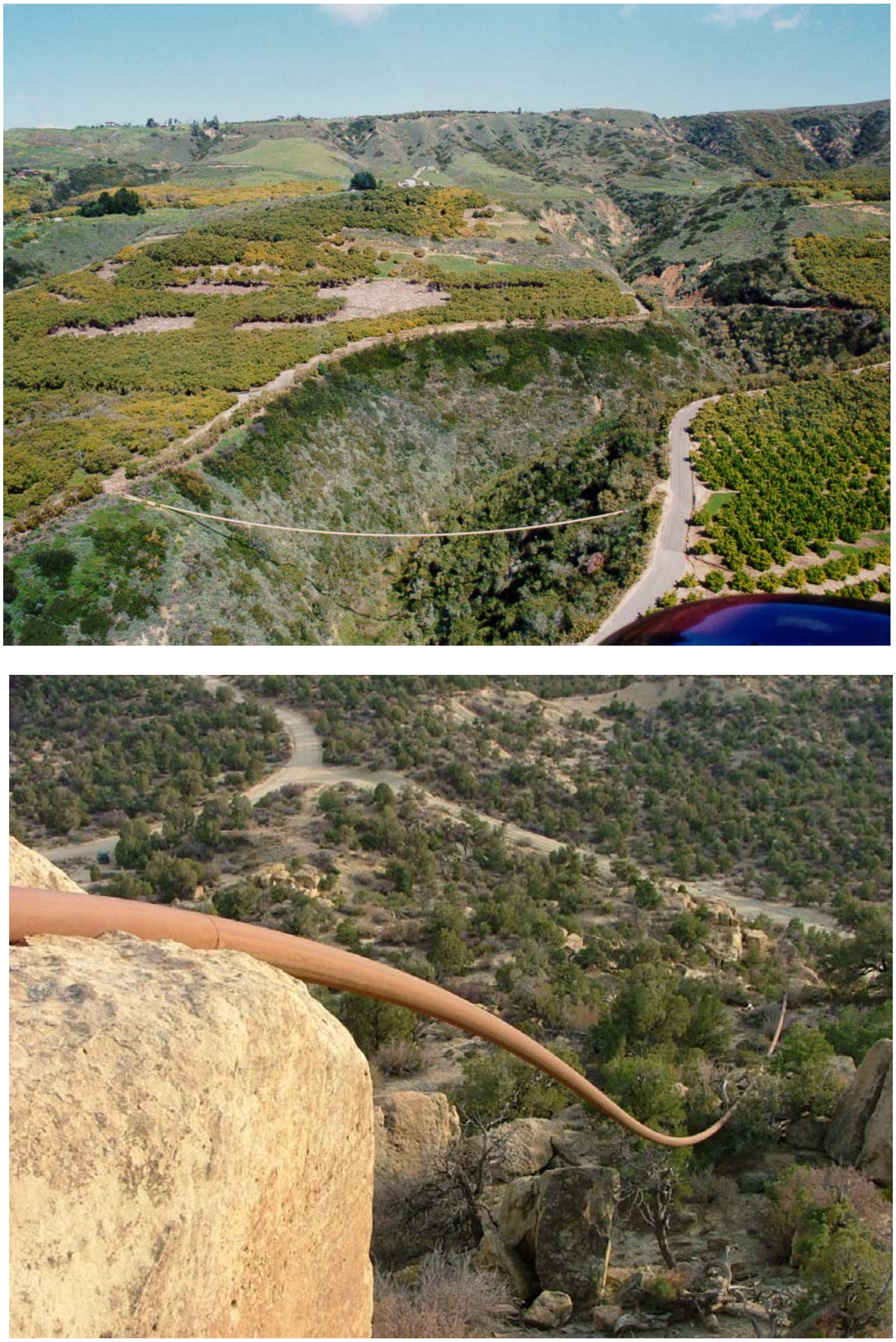


Figure 5.8 Examples of self-supported pipeline spans across difficult terrain

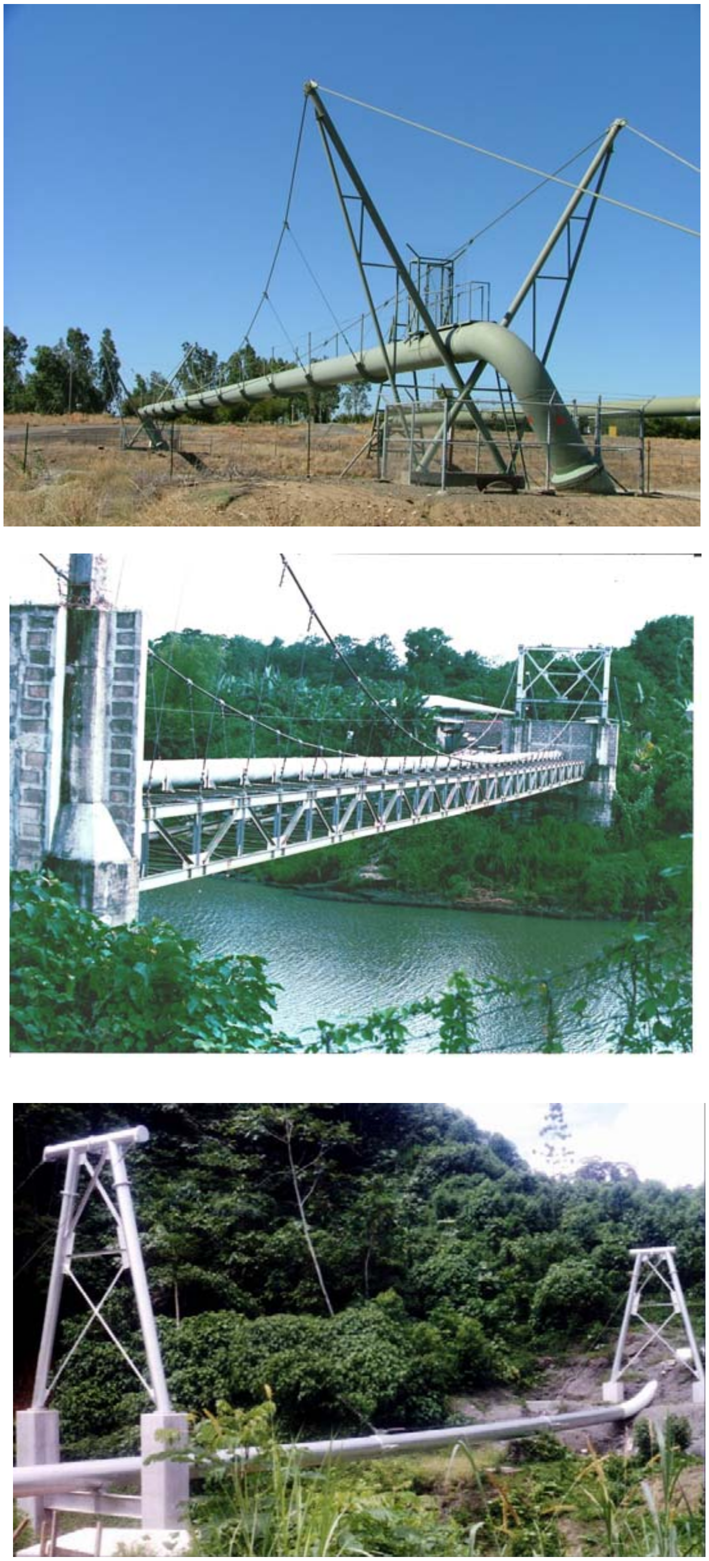


Figure 5.9 Examples of supported pipeline spans

Special precautions are necessary when installing pipelines beneath a slide zone using conventional trenching techniques to ensure that the backfill material does not become a means to convey surface or subsurface water, which might result in:

- Erosion of the trench fill and loss of pipe cover
- Increased pore water pressure within the slide mass that could activate deeper slide planes
- Increased water infiltration leading to slope instability at other locations on or near the pipeline right-of-way

6.0 GEOTECHNICAL MITIGATION ALTERNATIVES

Implementing geotechnical measures to eliminate or reduce the severity of potential landslide effects on a pipeline may be a viable alternative if pipeline design measures alone are determined to be insufficient or impractical. Generally, landslide mitigation measures have one or both of the following goals:

- Increase the stability of a slope, as measured by the factor of safety against sliding, to a level that the likelihood of future slope movement is acceptable
- Decrease the rate of slope movement to a level compatible with operational measures to manage the risk of adverse pipeline response

The construction of new slopes and stabilization of existing slopes are well-developed areas of geotechnical engineering practice and this guideline does not replicate presentation of design principles and analytical methods that are covered in detail in numerous other references.

The focus of this section is to acquaint the user with some of the more common approaches that have been constructed for pipeline applications and provide recommendations for the design factor of safety that is considered appropriate for different situations.

Geotechnical mitigation measures can be categorized into two basic approaches:

- Reduce the forces driving slide movement
- Increase the forces resisting slide movement

The various techniques for implementing these measures are provided in Table 6.1 along with some potential limitations on usage and requirements for implementation. Some brief discussion of the three basic geotechnical mitigation approaches is provided in sections 6.1 and 6.2. Additional discussion and numerous references to case-histories can be found in Turner and Schuster (1996). All techniques require a sound definition of the ground surface where a potential for slide movement exists and identification of the location of the critical slide plane at depth. In addition, all techniques involving reducing driving forces or increasing sliding resistance require characterization of subsurface soil strengths and groundwater conditions that can only be obtained through subsurface investigations with accompanying field and laboratory testing.

Past implementation of many of the measures in Table 6.1 has been largely associated with road and highway projects or commercial development of property where the site in question is under direct jurisdiction or ownership. Most of the measures in Table 6.1 will involve activities that extend outside of the typical pipeline right-of-way, which normally is less than 50 m, thus requiring agreements for a much wider right-of-way, obtaining approval from the landowner from whom the right-of-way is leased, or outright purchase of land adjacent to the right-of-way. This can pose a substantial challenge to identifying a practical geotechnical mitigation option for a pipeline project unless the landowner sees a direct measurable benefit from the mitigation option.

Table 6.1 Common geotechnical mitigation methods for landslides affecting pipelines

MITIGATION METHOD	LIMITATIONS	REQUIREMENTS
Remove Unstable Material	• May not be feasible because of right-of-way restrictions • May not be practical for large slides because of amount of material to be removed • May be difficult to implement while maintaining slide stability and pipe operation (if present) • May result in initiation of new landslide upslope • May not be feasible because of environmental restrictions	• Characterization of failure planes, soil strength, and groundwater conditions • Detailed stability analysis of conditions during and following removal of material
Bridge Unstable Area	• Feasible only for relatively small and shallow slide zones (suitable for existing pipelines)	• Characterization of failure planes and soil strength • Detailed structural analysis to assure bridge support members can withstand soil loads
Regrade Slope	• May not be feasible because of right-of-way and land use restrictions • May not be practical for large slides because of amount of material to be removed • May be difficult to implement while maintaining slide stability and pipe operation (if present) • May not be feasible because of environmental restrictions	• Characterization of failure planes, soil strength, and groundwater conditions • Detailed stability analysis of conditions during and following slope grading • Disposal of excavated soil and rock material • Assessment of impact of diverted surface water adjacent to pipeline right-of-way
Reduce Weight	• May not be feasible because of right-of-way restrictions • May not be practical for large slides because of amount of material to be replaced • May be difficult to implement while maintaining slide stability and pipe operation (if present) • May not be feasible because of environmental restrictions	• Characterization of failure planes, soil strength, and groundwater conditions • Detailed stability analysis of conditions during and following replacement of material
Reduce Surface Water Infiltration Reduce Groundwater Level	• May not be feasible because of the volume of water to be rerouted or removed and areas available for water discharge • Environmental restrictions may prevent discharge of drained subsurface water	• Characterization of failure planes, soil strength, and groundwater conditions • Estimates of surface water deposited by precipitation events and local drainage patterns • Detailed stability analysis of conditions before and after groundwater control measures • Assessment of impact of diverted surface water on adjacent to pipeline right-of-way • Monitoring and maintenance of drainage diversion mechanism • Assessment of groundwater level lowering on local wells
Buttresses, Counterweight Fill, and Toe Berms	• May not be feasible because of right-of-way and land use restrictions • May not be effective for deep-seated slides • Must be founded on firm foundation • May not be feasible because of environmental restrictions	• Characterization of failure planes, soil strength, and ground water conditions • Detailed stability analysis of conditions prior to and following construction • Estimates of surface water deposited by precipitation events and local drainage patterns

Table 6.1 Common Geotechnical Mitigation Methods for Landslides Affecting Pipelines (continued)

MITIGATION METHOD	LIMITATIONS	REQUIREMENTS
Structural Retaining Systems	• Rigid systems may not be able to withstand deformations • May not be able to be installed below sliding surface • Suitable for small slides only	• Characterization of failure planes, soil strength, and ground water conditions • Detailed stability analysis of conditions prior to and following construction • Estimates of surface water deposited by precipitation events and local drainage patterns
Anchors	• Foundation materials may not have sufficient strength to support anchor tension necessary to carry shear loads from sliding soil mass	• Characterization of failure planes, soil strength, and groundwater conditions • Detailed stability analysis of conditions prior to and following construction • Estimates of surface water deposited by precipitation events and local drainage patterns
In situ Soil Reinforcement	• Most effective for dense granular and stiff silty clay slopes • Long term integrity of reinforcement (soil nails, soil anchors, and piles) needs to be assured in permanent installations	• Characterization of failure planes, soil strength, and groundwater conditions • Detailed stability analysis of conditions prior to and following construction • Estimates of surface water deposited by precipitation events and local drainage patterns
Bank Armour	• Useful only at locations experiencing soil erosion from water flow leading to slope instability • Does not add to overall stability (unless armour mass is significant), only decreases soil loss rate	• Characterization of watercourse peak flows and direction • Characterization of soil loss and impact on slope stability • Impact of armouring on downstream areas (may increase bank or bed erosion at other locations)
Watercourse Flow Re-Direction	• May not be feasible because of land use and environmental restrictions • Useful only at locations experiencing soil erosion from water flow leading to slope instability • Does not add to overall stability, only decreases soil loss rate	• Characterization of watercourse peak flows and direction • Characterization of soil loss and impact on slope stability • Impact of flow re-direction on upstream and downstream areas (may increase bank or bed erosion at other locations)

6.1 Decrease Driving Forces

Decreasing driving forces on landslides involves altering the grade of the slope or controlling water pressure within the slide mass.

6.1.1 Regrade Slope

Slope modifications to decrease driving forces involve a combination of removal of material near the top of the slide zone and flattening of the slope. The amount of material to be moved can be determined using standard techniques to assess slope stability for various candidate grading concepts.

Because of the amount of material to be removed, modifying the grade of the slope is typically not applicable to slides characterized by long slopes with soil or rock overlying and parallel to more competent material. In such cases, the slip surface is constrained to the interface of the competent material and can be approximated by an infinite slope if the depth to the slip surface is small compared to the length of the slope. Similarly, grade modifications may be impractical if there is a considerable length of pipeline passing through a slide zone.

6.1.2 Reduce Ground Water Level

Reducing the ground water level within a potential slide mass is perhaps one of the most effective means to reduce the potential for slide movement. While drainage increases the effective stress in the soil mass which increases the shear strength of the soil, reducing the water level also substantially reduces the driving forces acting on the slide mass. There are a variety of methods for reducing the water level, with the selection of a particular method dependent upon the soil composition of the slope, local hydrology, and climatic conditions. For pipeline construction, the most commonly employed methods rely upon one or more of the techniques listed below and illustrated in Figure 6.1:

- Drainage blankets and trenches
- Drainage wells
- Horizontal drains

Drainage blankets, trenches, and wells rely upon replacement of poorly draining soil with free draining material. Drainage blankets are most practical in situations where a relatively thin layer of surface soil needs to be removed. As the depth of removed soil increases, the construction of single or multiple drainage trenches becomes more economical. Drainage wells are suitable for situations where the depth is uneconomical for drainage trenches. The size of drainage wells can be as large as 2 m in diameter and 50 m deep. If the local geology permits drainage wells to penetrate through a perched water zone in an unstable claystone or shale and into permeable fractured limestone, then the wells can drain by gravity and promote stability with construction being the only cost. However, if water must be pumped from the wells, then an on-going cost will be incurred, not to mention the need to have electric power to run pumps and some suitable place to discharge the produced water.

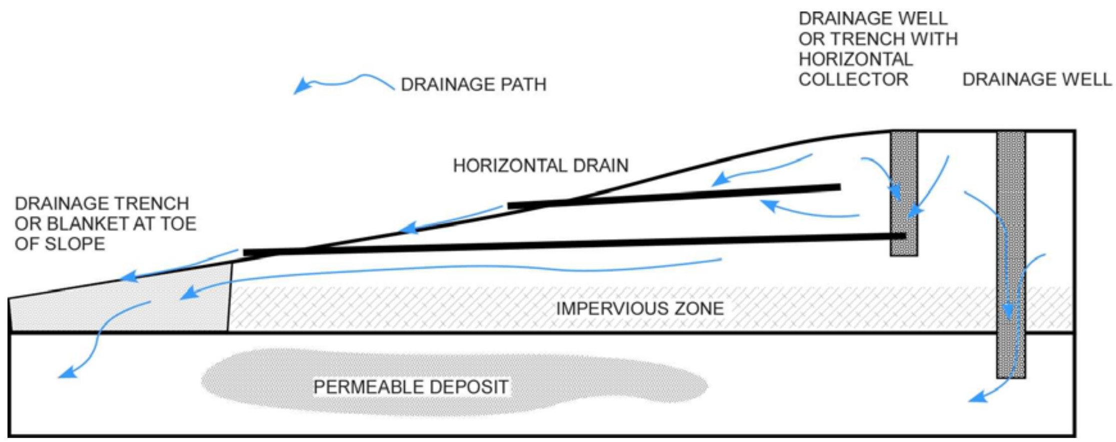


Figure 6.1 Schematic of typical subsurface drainage methods

Installation of horizontal drains, either alone or in conjunction with other stabilization methods, has been used extensively to provide groundwater lowering and control for pipelines and other linear infrastructure facilities (see Figure 6.2). Small diameter horizontal drains, typically ranging from 50 to 100 mm (2 to 4 inches) diameter can be installed rapidly and penetrate into existing or potential landslide areas for distances of the order of 30 to 150 m (100 to 500 ft.) and potentially more. Use of horizontal drains is of special benefit if adverse groundwater or geologic conditions are present at specific locations or depths within the slopes which would otherwise require extensive and costly alternative treatment.

In addition to providing drainage, the compacted fill used in the construction of drainage blankets, trenches, and wells can provide additional sliding resistance, albeit small, if they are composed of durable gravel or crushed rock and constructed so as to be keyed into competent material below the potential slide plane.

Other than the typical sizing and installation considerations, careful consideration needs to be given to the impact of water removal and additional water discharge on adjacent land owners or other portions of the pipeline alignment. Particular attention needs to be given to situations where runoff from drainage at one location on a slope causes erosion or increased pore water pressure downslope, leading to potential reactivation of a much larger slide mass.

Figure 6.2 Slope stabilization using horizontal drains, (A) Drain installation, (B) Drains installed along base of slope, (C)Seepage from drains, (D) Routing of drained water, (E) Finished installation

In addition to the subsurface groundwater control measures described above, collection or diversion of near surface seepage and runoff or channel flows can often provide significant lowering and control of groundwater levels, in particular for existing or potential shallow landslides overlying competent materials. Installation of upslope interceptor ditches or drains, with or without use of relatively impermeable linings or flumes along the ditch invert, is commonly used for groundwater and runoff control for highway construction. Similarly, within existing or potential deep seated landslides, surface drainage measures have been used to drain depressions, such as sag ponds, or divert surface runoff concentrations around or across broken terrain and fissures within the landslide mass.

In some jurisdictions, environmental regulations or legal liability can place substantial restrictions on the discharge of drained subsurface water. Some geologic settings include formations that can contaminate subsurface water. For example, black shale, with pyrite and other sulfide minerals can oxidizes to produce sulfuric acid drainage and liberated metal ions which are hazardous in the environment.

6.1.3 Replace Slope Material with Light-Weight Material

Driving forces can be substantially reduced by replacing slope material with light-weight fill such as polyethylene or polyurethane foam, commonly referred to as “geofoam,” wood chips, shredded tires, expanded clay or shale aggregate, etc. While the development of many of the applications for light-weight fill material have been directed toward the construction of embankments over weak or compressible in-situ soil for the highway and forestry industry, the applications are equally applicable to mitigating potential slope movements. The primary obstacles to light-weight slope material placement are related to the amount of material that needs to be moved and the level of alteration to existing water drainage patterns and the cost.

While geofoam replacement is perhaps one of the more expensive solutions, it can provide an additional advantage in that much steeper slopes can be reconstructed than would normally be achievable with other materials.

However, with incorporation of geotextile or geogrid reinforcement, other potentially less costly light-weight materials (such as pumice, expanded shale, shredded or recycled tires, chipped woodwaste, etc.) can also be utilized to develop steep, but light-weight slopes.

6.1.4 Remove Unstable Materials

For well-defined shallow soil slides over competent material and extending along a limited length of a pipeline alignment, it may be feasible to simply remove the potentially unstable material. This approach differs from changing the slope topography, as discussed in section 6.2.1, in that there is no exposure to a landslide hazard following the removal of unstable material. The need to remove the full length of unstable material should also be assessed. If a sufficient length is removed so that any future soil movement is not injurious to the pipeline, this may be sufficient from a pipe integrity perspective but may not eliminate environmental consequences.

The economic feasibility of this approach is dependent upon several factors:

- Amount and depth of material to be removed
- Stability of the material upslope of the landslide
- Ability to readily dispose of removed material (i.e., a suitable location and no hazardous materials, such as naturally occurring asbestos or acid-generating pyrite)
- Measures necessary to prevent potential adverse impacts on existing surface or subsurface drainage patterns
- Measures to minimize or offset aesthetic impacts of material removal (e.g., visual barriers, matching color and texture of surrounding terrain)
- Measures to minimize or offset environmental impacts associated with the removal process (e.g., habitat restoration requirements, monitoring of adjacent streams or wells)

The technical feasibility also depends on the width of the unstable material and how removal at the pipeline right-of-way may impact adjacent land use or landowners. In most cases, slide hazards that are amenable to mitigation through removal of unstable materials are also candidates for installing the pipeline below the unstable area as discussed in section 6.1.3.

6.2 Increase Resisting Forces

Resistance to potential slide displacements can be increased by both external and internal slope strengthening measures. However, reinforcement of slope materials is generally practical only for relatively small slopes with shallow failure planes.

6.2.1 Reduce Ground Water Level

As noted in section 6.2.2, reducing ground water levels not only reduces the driving forces associated with the weight of the slope material but it also increases soil shear strength by increasing the effective vertical stress in the soil mass.

6.2.2 Provide External Resistance

Methods of providing external resistance include increasing the sliding resistance at the toe of a potential slide zone and various types of anchoring systems.

Increasing Resistance at the Toe of a Slide

Methods to increase the sliding resistance at the toe of a slide zone include the placement of additional fill material and construction of earth retaining structures. The general principles of both of these broad categories of toe reinforcement are well established in geotechnical engineering and only a brief summary of various options is presented.

The placement of fill at the toe of a slide zone provides additional sliding resistance through a combination of fill weight and the shear strength of the fill material. Commonly referred to as buttresses, counterweight fills, or toe berms, the design of the fill should assure adequate sliding resistance within or below the base of the fill, preventing overturning, and assuring adequate foundation bearing capacity. The berms typically require a substantial width parallel to the slope in order to stabilize the slope. Local land-use may restrict the size of the berm and thus its effectiveness. These toe measures require space for construction and a sequence in which material is removed, stockpiled, and brought back after the shear key is constructed with suitable free-draining crushed rock or durable gravel deposits.

Earth Retaining Structures

When providing fill at the toe of a slope is impractical because of limited right-of-way or other land use constraints, earth retaining structures may be a feasible alternative. Earth retaining structures can generally be categorized as externally and internally reinforced structures. Externally reinforced systems include rigid cantilever walls (e.g. Vasconcellos et al., 2004), gravity systems that rely upon weight as the primary external resistance, tie-back systems employing soil or rock anchors, and vertical piles or caissons.

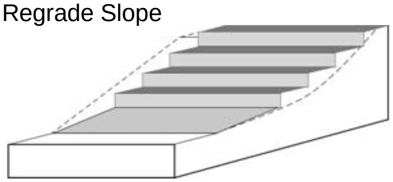
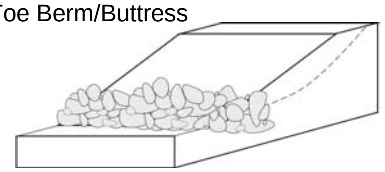
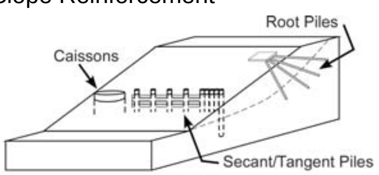
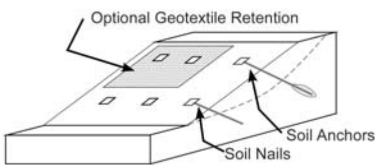
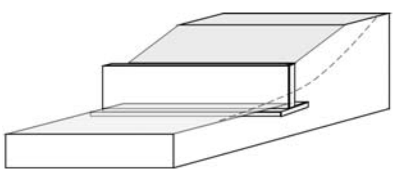
Internally reinforced systems can be categorized in terms of applications to existing slope conditions and applications for constructed slopes or embankments. Constructed slopes and embankments can accommodate various types of soil reinforcement into the backfill including geotextiles and metallic or polymeric grid, meshes, and strips.

6.2.3 Reinforce Existing Slope

Existing slope reinforcement generally involves installation of a series of rod-like members that that increase the resistance to slope movement through the shear strength of the members themselves or are anchored into competent rock or soil and provide direct resistance via attachments to anchor plates or facing plates. Types of reinforcement include soil nails, soil and rock anchors, micropiles, pin piles, and root piles.

Table 6.2 provides a summary of the various methods, their advantages and disadvantages, which can be considered to improve the stability of existing or potential landslide areas. As described previously, combinations of these methods, in particular inclusion of drainage control measures, are often required to achieve the necessary or optimum improvement in stability.

Table 6.2 Advantages and disadvantages of various geotechnical mitigation options

Method	Mechanism	Advantages	Disadvantages
Regrade Slope 	Reduce driving forces	Can be carried out using conventional earth moving equipment. Can be combined with construction of toe berm/buttress. Can provide access for other measures such as slope reinforcement, installation of drainage measures	Detailed knowledge of subsurface conditions and critical failure plane required. May require excavation and disposal of large volumes of material. Not generally suitable for deep seated landslides or long slopes above pipelines. Requires care to prevent instability during regrading May require significant regulatory approvals, agreements with neighboring property owners and extensive revegetation and landscaping/environmental mitigation.
Toe Berm/Buttress 	Increase stability at toe	Can be carried out using conventional earth moving equipment. Can be combined with regrading	Detailed knowledge of subsurface conditions and critical failure plane required. May require excavation and placement of large volumes of material. May require significant regulatory approvals, agreements with neighboring property owners and extensive revegetation and landscaping/environmental mitigation.
Slope Reinforcement  	Increase shear resistance along slide plane	Can be used to provide improved shear resistance along one or more slide planes, and at various depths not treatable using slope regrading and toe berm methods	Detailed knowledge of subsurface conditions, critical failure plane, as well as expected direction of landslide movement required. Slope reinforcement typically requires use of specialty contractors. Installation of slope reinforcement above or below pipeline alignment may result in disturbance of slope and increased landslide risk during installation, as well as significant regulatory approvals, agreements with neighboring property owners and extensive revegetation and landscaping/environmental mitigation.
Retaining Structure 	Provide external resisting force	Can permit development of effective toe buttress support within restricted pipeline corridors Can be combined with regrading	Detailed knowledge of subsurface conditions and critical failure plane required. May require use of specialty contractors. Not generally suitable for deep seated landslides since foundation of retaining structure must be located below or outside failure plane. Requires care to prevent instability during excavation for foundation construction at toe of slide/ May require significant regulatory approvals, agreements with neighboring property owners and extensive revegetation and landscaping/environmental mitigation.

7.0 OPERATIONAL MONITORING & MITIGATION OF LANDSLIDE HAZARDS

In addition to design measures discussed in sections 5.0 and 6.0, operational measures can be implemented to reduce the likelihood of severe damage or the consequences of damage. Operational mitigation measures can generally be categorized as passive or active based upon whether or not the measures require reaction to changing conditions (active) or reaction after landslide movement has begun and some pipeline damage has occurred (passive). Active measures may include such items as strain relief, pressure reduction (likely only a temporary measure), pipe isolation, relocation, and others. Passive measures may include such items as valves, increased separation, barriers, and others.

Active measures require some type of monitoring of the pipeline, of ground displacement, of local area factors (rainfall, land use change, groundwater changes, etc.) or all three. This monitoring provides the basis for determining when actions need to be taken to ameliorate unacceptable conditions along the pipeline or obtain more detailed information on the nature of the ground displacement hazard. The most common type of active operational measures employed for pipelines involve the following types of activities:

1. Implement a monitoring plan for the pipeline and/or the zone of ground displacement and/or local area factors.
2. Establish threshold levels (i.e. alarm, mitigate, critical) for ground displacement or pipeline strain.
3. Execute existing plans, which may include pre-positioning of materials and equipment, to relieve pipeline strains and to reduce ground movement rates or its effect. Relief of pipeline strain will typically involve a combination of removing soil cover to allow high localized pipeline strains to be distributed as a reduced level of strain increase over a longer length of pipeline, repositioning a pipeline to the position held prior to ground movement, and cutting the pipeline to release locked-in strain.

The methods and levels of monitoring necessary to support active operational mitigation are highly variable and will depend upon an assessment of the relative likelihood and severity of ground displacement. More monitoring locations and more frequent monitoring observations will generally be needed where ongoing ground displacements are known or suspected to exist. This need may justify the installation of permanent survey monuments or instrumentation such as slope inclinometers and piezometers. It may also justify continuous remote monitoring as opposed to periodic or campaign monitoring. Slopes that are not active, but judged to be potentially active, are more likely to be surveyed intermittently (once or twice a year) or following environmental events (such as a heavy rainfall or significant spring snow-melt or a significant land use change) using remote methods.

As previously discussed in this guideline, current technology is generally unable to accurately predict the frequency or severity of future slope movements with a level of reliability typically desired for natural gas or liquid hydrocarbon transmission pipelines. Thus, the most significant challenge in assessing the effectiveness of monitoring and implementing an active operational mitigation measure is assigning the probability of successfully responding to adverse ground displacements before significant pipeline or environmental damage has occurred. This is particularly true for pipelines located in remote regions or regions where year-round access may be limited by weather or ground conditions. Both of these restrictive conditions will typically exist for Arctic pipelines. The uncertainty in the effectiveness of active operational mitigation measures can be offset in some cases by having a high degree of certainty in the knowledge of the level (length and depth) of ground displacement that can occur before the pipeline does sustain significant damage. If the 'worst case' movement cannot significantly damage the pipe, then there is less urgency in an intervention occurring in a timely manner. It must be kept in mind though, that even in these cases, a large ground movement may result in a reduction in operating pressure or even a pipe shut-in, resulting in delivery interruption. A large ground movement may also result in significant environmental damage, which may also be a potential large liability.

The option also exists to "pre-implement" a pipeline design measure to increase the level of ground displacement that the pipeline can withstand prior to sustaining damage. This increases the likelihood that the adverse condition will be identified and corrected. This does require a monetary investment which needs to be offset against the potential benefits of reduced risk.

Passive operational mitigation measures will generally accept potential pipeline damage from ground displacement and focus on reducing the consequences (third party and environmental) of such damage. Examples of methods to reduce consequences include the following:

- Increasing stand-off distance through additional land procurement and access control measures
- Constructing protective barriers to minimize thermal exposure in the event of product ignition
- Installing line-break valves to limit loss of pipeline contents and duration of hazardous conditions in the event of pipeline damage
- Constructing dikes, drainage channels, and basins to contain spills from liquid lines
- Pre-positioning of materials and equipment to facilitate rapid repair to minimize duration of service outage

For new pipelines, the viability of strictly passive operational mitigation measures is practically limited to very remote or largely uninhabited regions where the existing population is low, there is little prospect for future development, the costs of procuring additional property are not prohibitively expensive, and the likelihood of substantial ground displacement is low. Furthermore, application of passive measures for liquid hydrocarbon pipelines is likely only feasible where there are minimal ecological resources that might be adversely impacted. Obtaining governmental agency approval for construction and operation of a new pipeline with a recognized possibility for damage resulting in loss of pressure integrity may be difficult and time consuming within typical time constraints.

Decisions regarding what types of operational strategies to manage risks from landslides and subsidence are typically a portion of an overall pipeline risk management process that addresses a wide variety of pipeline integrity issues (e.g., third-party damage, corrosion, erosion, etc.). An example of the processes by which decisions are made as to whether it is acceptable to employ some type of monitoring or implement mitigation measures is illustrated in Figure 7.1.

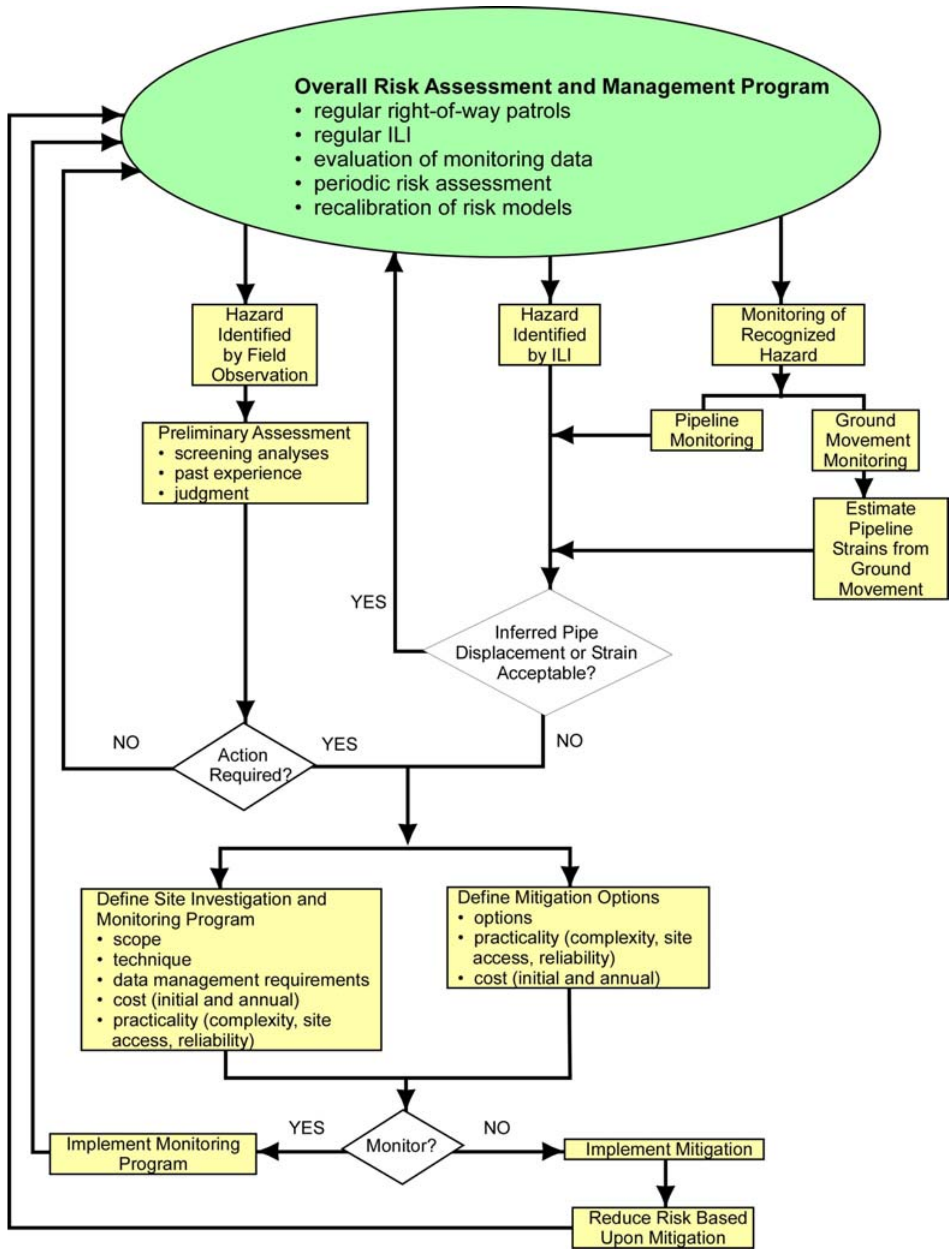


Figure 7.1 Decision process to determine need for monitoring or mitigation measures

The starting point for the process in Figure 7.1 is the overall pipeline risk management process that is assumed to include regular alignment patrols, scheduled in-line-inspections (ILI), and collection and assessment of data from existing pipeline or ground movement monitoring locations along the pipeline. With these assumptions, discovery of new potential ground movement hazards will result from patrol observations or in-line inspection data.

Data collected through in-line inspection at monitoring locations can provided direct information on the amount of strain that has been induced in the pipeline or the amount of deformation the pipeline has experienced, which can be used to determine pipeline strain through appropriate soil-pipeline interaction models. If the pipeline condition is not considered acceptable, a decision must be made to either increase the level of monitoring by installing additional instrumentation or increased measurement frequency, or take action to mitigate the effects of the hazard on the pipeline. The decision to perform additional monitoring is only acceptable when the risk to the pipeline is considered acceptable during the time period necessary to collect additional data.

It is common to consider two levels of strain limits beyond normal operation conditions. A lower level strain limit is usually associated with detailed site investigations and installation of monitoring equipment. An upper level strain limit is established as a trigger to initiate the planning and implementation of mitigation measures. Establishing acceptable strain criteria can be challenging, particularly when addressing existing pipelines that may have some level of deteriorated condition. Setting strain criteria also needs to consider the time to install, or otherwise implement, additional monitoring measures and the probability of success in obtaining additional data or arresting the impacts of the underlying geohazard on the pipeline.

The decisions to react to potential geohazard problem sites identified through patrols (ground, aerial, or satellite) by undertaking site investigations and installing monitoring instrumentation or undertaking mitigative actions should be guided by an engineering assessment based on screening analysis and reasonable parameter values. With respect to potential geohazard induced pipeline integrity issues, it is important to fully consider the ground displacement capacity of the pipeline as opposed to undertaking mitigation studies with a purely geotechnical focus.

An essential step in the overall geohazard management process is updating the risk assessment database with additional monitoring information and recognize the risk reduction benefits where the mitigative measures have been implemented. The risk reduction from mitigation should incorporate an assessment of the probability that the installed mitigative measure was partially or fully unsuccessful.

7.1 Pipeline Monitoring

7.1.1 Strain Gages

The most common type of pipeline monitoring consists of installing instrumentation on the pipeline to measure strains developed as the pipeline responds to ground displacement.

Strain gages, either foil or vibrating wire, welded to the exterior of the pipeline are the most common type of installation (Figure 7.2). In addition to conventional strain gages, fiber-optic sensors, in linear, coiled, or mesh configurations (see Figure 7.3) are available that can provide strain measurements over a long length of pipeline or around the pipeline circumference.

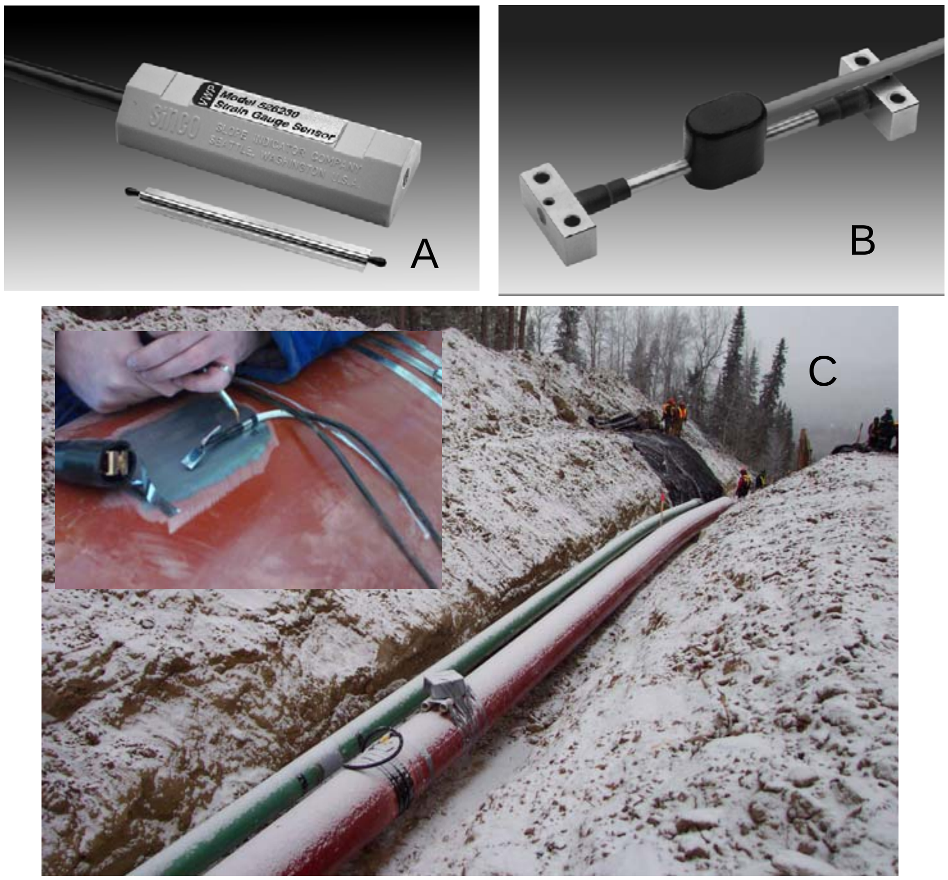


Figure 7.2 Exampled of typical strain gage instruments, (A) Vibrating wire strain gage slope indicator, (B) Vibrating wire strain gage displacement gage, (C) Welded foil strain gage installation and completed strain gage location with read-out-box

Strain gages installed during pipeline construction have one significant advantage over an internal measurement tools; they can provide data on stress changes from pipe lowering-in, through backfilling, pipe wall temperature changes, and finally pipe start-up.

Key challenges to monitoring pipeline strain with strain gages include the following:

1. Prior to any indications of ground displacement, it is difficult to impossible to identify the location along the pipeline and around the pipe circumference for installation of strain gages that corresponds to the location of highest strain. This difficulty is only slightly reduced once the ground displacement pattern along the pipeline becomes evident.

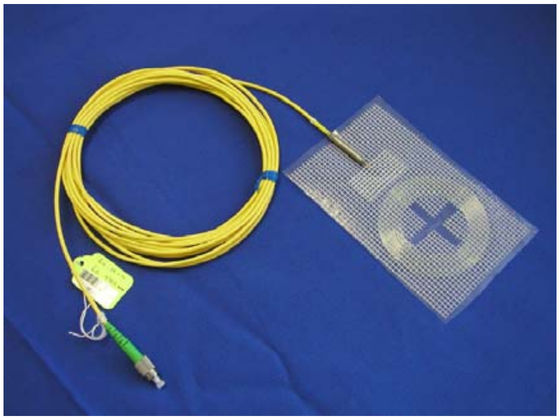


Figure 7.3 Exampled of fiber optic strain gage and installation of linear and local fiber optic strain gages on a pipeline

2. Installation of gages on an operating line requires excavating the pipe and corrosion coating removal. This may involve lowering the operating pressure during strain gage installation.
3. Costs associated with installation and long-term monitoring and maintenance of the measurement system can be considerable, especially for multiple instrumented locations or installations in remote areas where some type of telemetry system is necessary for continuous or regular collection of measurement data. As with many monitoring methods, the initial capital costs of procuring and installing the measurement system is a fraction of the costs associated with long-term operation and maintenance of the system.
4. All measurement systems installed on buried pipelines are subject to harsh environmental conditions, requiring the installation of redundant instrumentation or periodic replacement of instruments. Initially installing a large number of redundant gages is often more cost effective than re-excavating to inspect and install replacement gages.
5. The effectiveness of the pipeline monitoring system is dependent upon selecting the appropriate location along the pipeline and around the pipeline circumference. Gages must be installed at a minimum of three points around the pipe to capture axial and bending components of pipe strain. An analysis of pipeline response to expected ground displacement using the approach described in section 5.0 can provide an indication of the pipeline locations likely to experience the largest strain. However, there will typically be some level of uncertainty related to the agreement between the ground displacements used in the analysis and actual ground displacements, and thus the areas of highest pipe strain.
6. Analysis of strain gage data is not always straightforward. It must be done by experienced personnel and ideally, by the same firm that installed the gages, thus it is important to hire a qualified firm from the beginning. Also, it is important to

remember that the strain gages will only detect strain changes from the time they were installed. Any preexisting strain must be considered in the overall strain assessment.

7.1.2 Blind Hole Drilling

Blind hole drilling (ASTM E-387) is a means to assess the elastic component of the residual or ambient strain in the pipe wall of an operating pipeline. A small diameter, shallow hole is drilled in the pipe while multiple strain gage elements are used to monitor the change in strain as the hole is advanced. The strain measurements can be directly related to elastic components of in-plane stresses in the pipe wall.

Stresses determined from blind hole drilling will be sensitive to residual stresses from the pipe manufacturing process as well as any stresses imposed by ground movement. Residual stresses from manufacturing are typically not considered to have a detrimental effect on the pipeline strain capacity available to respond to ground movement. However, residual manufacturing strains will not be able to be differentiated from strains resulting from other conditions. In addition, blind hole drilling can not determine the level of stress above the linear portion of the stress-strain curve for the pipe material. Therefore, the value of blind hole drilling is considered very limited and essentially nil where strain-based criteria are used as the basis to ascertain acceptable pipeline deformations under conditions of imposed ground displacement.

7.1.3 In-line Inspection

Some specialized in-line inspection (ILI) tools, often referred to as geometry pigs, are used to map the centerline of a pipeline. Geometry pigs are capable of measuring pipe centerline orientation (pitch and azimuth) and odometer distance. From these measurements, the northing, easting and elevation coordinates and the vertical, horizontal and resultant curvature and the induced flexural strains can be deduced.

Hart et al. (2008) developed an algorithm (see Appendix B) for deducing the longitudinal or axial strain from geometry pig measurements of a laterally displaced pipeline. The algorithm is limited to lateral displacements of the pipeline that results in a predominantly transverse loading; i.e., the induced transverse component of the loading is much greater than its axial component. The algorithm is generally capable of determining pipeline longitudinal strain within ±10% to ±20%.

The approach in Hart et al. (2008) is based upon changes in the pipeline geometry and in particular, changes in the pipeline curvature. In long, straight sections of real pipelines remote from field and fabricated bends, the actual profile of the pipeline will not be perfectly straight. There are inevitable variations in the trench profile which will result in modest amounts of elastic “roping” curvature over distances of one to several pipe joint lengths. In addition to these “global” deviations from a perfectly straight pipeline profile, a geometry pig survey will undoubtedly highlight repeatable, low amplitude, short length noise features along the pipe profile (e.g., due to expander marks, longitudinal weld seams, weld beads, minor offsets and misalignments at girth welds, etc.). Although all of these features of the “as-built” pipeline geometry can show up as curvature in the data from a geometry pig survey, none of these features are associated with pipeline curvature due to imposed ground displacements. Therefore, the ideal framework for applying the approach of Hart et al. (2008) is one in which there is a baseline geometry pig survey of the pipeline soon after construction. The lack of a baseline survey can be an impediment to the accuracy of strains deduced from curvature measurements. However, the loss of accuracy is generally no greater than any other approach, including various external gages and instruments and analytical modeling, that rely upon assumptions regarding the as-built pipeline geometry.

Another key factor in determining longitudinal strain from geometry pig data is the gage length over which numerical differentiation of the pipeline orientation is performed. Specifically, selecting too large of a gage length will result in the curvature being underestimated. Establishing a reliable estimate for the curvature is further complicated by noise in the geometry pig data signals resulting from pipeline irregularities, girth welds, etc. Various means for filtering the geopig data are available, although each has some potential drawbacks. For example, one way that is often used to smooth out the data is to select a gage length that is several multiples of the pipe diameter (longer than the characteristic length of the noise features). This is a feasible approach provided that the gage length does not degrade the accuracy of the curvature measurement (i.e., the gauge length must be small compared to the length of the pipeline undergoing bending deformation). Therefore, care must be exercised when selecting an appropriate gage length and it may even be appropriate to evaluate the curvature using two different gage lengths. As noted, a baseline survey of the "as built" pipeline is required to deduce pipeline strains from ground deformation following construction. The baseline survey can also be used to identify and eliminate measurement noise traceable to pipeline irregularities.

7.1.4 On-Pipe Survey Monuments

This method of pipe monitoring consists of attaching a metal plate with a central peak to the crown of the pipe. A series of plates are located along the pipe within the zone of ground movement. The plate center points are surveyed in using highly precise surveying with a large number of redundant surveys. Dry wells are then placed over the pipe and plates and usually covered with a locked lid to allow access for subsequent surveys.

Translation and rotation of the pipe monument from subsequent surveys can be used to estimate the variation of pipe strain. The results can be used on their own or can provide for the targeted placement of strain gages, and thus a more accurate analysis of pipe strain.

The disadvantage of the method is that the surveys and data analysis are fairly expensive, although recent advances in surveying may reduce the field time required.

7.2 Landslide Monitoring

This section discusses different monitoring styles and various measurements, techniques, and kinds of instruments available for each. The discussion is focused on landslide hazards as many of the techniques were developed to address these hazards. However, methods related to measurements of the ground surface are generally applicable to monitoring displacements related to subsidence.

Monitoring and instrumentation have several applications in the assessment of landslide hazards: (1) to obtain parameters and dimensions for stability and deformation analysis, (2) to observe the performance or stability of a slope, (3) to help identify the spatial limits of the area that is moving, (4) and to provide notification of renewed or accelerated movement. The cost and complexity of instrumentation limits the practical use of long-term monitoring and instrumentation to investigations of large, complex ground displacement patterns. It should be noted in comparison to monitoring pipelines as discussed in section 7.1, deformation of the ground typically begins to occur at a detectible scale before sufficient displacement has occurred to the pipeline to be detected by strain gages. Therefore, earlier indications of potentially damaging deformation may be more readily realized from landslide monitoring than from pipeline monitoring.

The applications of monitoring and instrumentation methods rely on several kinds of measurements. Instrumentation is needed to obtain parameters for slope stability analysis and numerical modeling of known landslide areas or those areas where mitigative or remedial measures to reduce risk of future landslides are being considered Monitoring of pore water pressure can also identify threshold pore pressure needed to induce movement and provide early warning of impending movement (Picarelli and Russo, 2004; Ellis et al., 2007). Most other instrumental monitoring of landslides is concerned with observing and characterizing the displacement and deformation of the landslide mass. Precipitation is commonly recorded as well to observe any connection between precipitation and landslide movement (Mikkelsen, 1996; Baum and Reid, 1995). Precipitation is a common trigger of landslide movement. However, a minimum amount of precipitation needs to fall in a season to fulfill an antecedent condition before the slope system is susceptible triggered movement from a single storm.

Landslide monitoring can be classified into three different styles or types based on the frequency and mode of measurement: (1) campaign, (2) continuous, and (3) real-time. Campaign-style monitoring consists of a series of repeated surveys and measurements at established monitoring points or repeated acquisition of remotely sensed imagery. Consequently, this style of monitoring is generally the least frequent and is amenable to simple measuring devices as well as sophisticated instruments. Campaign monitoring includes the classical methods of landslide monitoring, such as repeated surveys of landslide movement and subsurface water levels, and new methods that use laser scanners or satellite remote sensing. The primary strength of campaign monitoring is the ability to determine the spatial variability of conditions, such as movement or water level, in landslides. With the exception of remote sensing methods, campaign-style monitoring requires regular visits to the field site.

Continuous monitoring relies on instruments and equipment to record measurements continuously or at regular closely spaced intervals and to save the measurements at the site for later retrieval.

Real-time (more correctly, near-real-time) monitoring combines continuous monitoring with some form of automated telemetry and data processing so that monitoring results from a remote site are available to project engineers, emergency response personnel, or others within a short time after the actual measurements occur. Real-time data processing may occur on site or at the project office.

Continuous and real-time monitoring both require periodic visits to the field site for instrument maintenance or repairs; continuous monitoring also requires regular visits to collect the stored data.

Different landslide monitoring techniques provide measurements of rate, direction, and amount of movement or deformation, landslide depth, landslide extent (plan view dimensions), subsurface water conditions, and earth pressures. Each of the three monitoring styles includes a range of available techniques for making different kinds of measurements. Most landslide monitoring projects require a combination of different styles of monitoring to adequately characterize movement and conditions that induce landslide movement. For completeness, brief reference is made in the following sections to long-established (pre-1996) monitoring techniques. Techniques that have appeared since the publication of Transportation Research Board Special Report 247 (Turner and Schuster 1996) are described briefly.

7.2.1 Displacement and Deformation

Several styles of campaign monitoring exist for determining displacement and deformation as described briefly in the following paragraphs. Tables 7.1 and 7.2 provide additional description and comparison of the methods.

A large range of instruments and techniques exist for making time-series measurements of landslide displacement and deformation (Table 7.3). Although the majority of these techniques are designed for monitoring points on the ground surface, at least one remote sensing technique for monitoring changes over an area is adapted to continuous monitoring and several subsurface techniques of strain and displacement monitoring are available as well. Each technique listed in Table 7.3 has certain advantages and drawbacks.

For example, wire extensometers are relatively inexpensive and reliable, but wind, animals, and other environmental factors can cause false deformation readings unless the extensometer cable is adequately protected. Most devices for continuous displacement monitoring must occasionally be reset or realigned after an amount of movement that varies with the nature and measurement range of the instrument. Others, such as borehole inclinometers and coaxial cables for TDR, are destroyed when displacement exceeds the range of the instrument. Most available techniques have been described and evaluated elsewhere (Mikkelsen, 1996; Keaton and DeGraff, 1996). However a few new techniques are worth describing further.

Point and Line Surveys

Regardless of technique, these surveys attempt to determine the change in position of points on the ground surface of the landslide. Originally these surveys were performed using a steel measuring tape, tape extensometer, or conventional surveying instruments to determine the positions of known points on the landslide relative to fixed points on stationary ground (Keaton and DeGraff, 1996). Keaton and DeGraff (1996) have compiled a more detailed evaluation of various conventional and modern surveying techniques for use in geologic mapping and monitoring of landslides.

Keaton and Gailing (2004) describe a quadrilateral system installed between a pipeline bridge foundation and the crest of a slope below which a landslide had occurred. The quadrilateral approach was described by Baum et al. (1988). Each quadrilateral is an array of four points defined by six distances and relative elevations of the points. The four points define four triangles. Differences in distances and relative elevations between readings are used to calculate displacements, strains, and tilts. Keaton and Gailing (2004) used two-inch-diameter steel pipes that extended approximately six feet into the ground and ere filled with grout. Distance measurements were made with a tape extensometer and elevation measurements were made with an optical survey level. Calculations are based on Lagrangian interpolation and done with an electronic spreadsheet program.

Terrestrial Laser Scanning

The recent advent of laser scanning, also referred to as terrestrial lidar or tripod lidar, opens new avenues for time-lapse surveys. Laser scanners are capable of rapidly measuring and recording locations of millions of closely spaced points on the ground surface. Laser scans, such as those used to generate the maps in Figure 7.4, make it possible to image the surfaces of landslides and unstable hillsides (Rowlands et al., 2003; Jones, 2006; Collins et al., 2007).

Differencing scans taken on different dates reveals changes that result from landslide deformation and redistribution of materials. Displacement can also be computed for features or markers that are identifiable in imagery from successive scans. Distance accuracy ranges from 1 cm to 5 cm for rapid, long-range scanners that are suited to topographic surveying to millimeters for slower, short range scanners that are designed for detailed scanning.

Despite the exciting possibilities offered by laser scanning, certain disadvantages hinder widespread application to routine landslide monitoring. These are high equipment and software costs, a steep learning curve, and the large amount of time required to process data after acquisition. Application of laser scanning to continuous or real-time monitoring is limited to repeatedly scanning a small area of several square meters from a fixed point (Bawden, oral communication, 2007). Although this might be adequate for a small landslide or small area of particular concern, repeated scanning of selected more widely scattered points using a robotic total station might be more effective on a larger landslide.

Aerial Imaging and Satellite Remote Sensing

Much recent work has been devoted to detecting landslides and determining landslide displacement from remotely sensed data (Van Westen, 2004; Farina et al., 2006). High-resolution aerial photography and synthetic aperture radar (SAR) imagery have been the most widely used for determining displacement (Table 7.2). All of these techniques use imagery acquired on different dates to determine landslide displacement based upon positions of key features in successive images or calculated on a pixel-by-pixel basis for the entire scene. Additional details regarding the use of satellite radar imaging is provided in Appendix C.

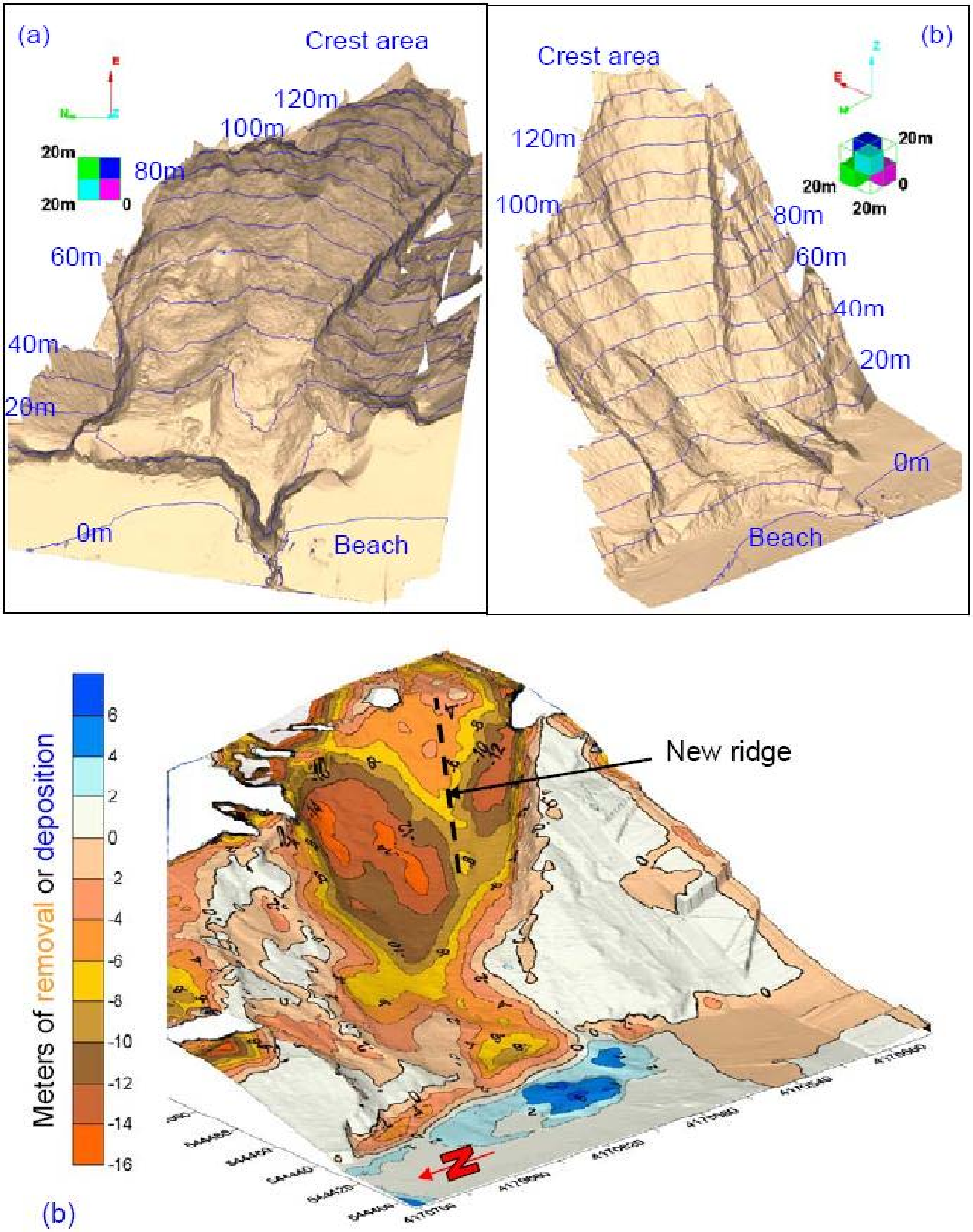


Figure 7.4 Example images of landslide area obtained by laser scanning (A), shaded relief and contour map, plan view, (B) Oblique view, (C) Difference plot showing changes in elevation that resulted from landslide movement (Collins et al., 2007)

Table 7.1 Comparison of equipment and resolution for repeat ground surveys to determine surface displacement

Technique	Description	Resolution	Advantages and drawbacks	Initial Cost[1]	SCR[2]	Sites[3]	Data[4]	Reference
Tape	Measure distances between stakes arranged in grids, quadrilaterals or other configurations to determine displacement.	Sub-decimeter to centimeter resolution, depending on distance, ground surface irregularities, and other factors	Simple, reliable, inexpensive technology Stakes subject to tilting or other disturbance, provides only 1-D displacement, manual readings contribute to increased frequency of errors	Low	1:10	S	S	Baum et al., 1988; Keaton and DeGraff, 1996
Tape extensometer	Measure distances between fixed points; used with quadrilaterals to calculate displacement, strain, and tilt	Millimeter	Simple, reliable technology Requires specialized monuments	Low	1:10	S	S	Mikkelsen, 1996 Keaton and Gailing, 2004
Total station	By means of standard surveying techniques, establish 3-dimensional locations of points on the ground surface.	Centimeter to sub centimeter, depending on environmental conditions and skill of operator.	Provides accurate three-dimensional displacement of discrete points. Some units capable of measuring hard-to-reach points without a prism.	Low to Moderate	1:1	M	M	Keaton and DeGraff, 1996
Differential GPS	Uses fixed GPS base station and roving receivers to acquire position data at selected points	Sub centimeter horizontal, sub-decimeter vertical if stations occupied at least 20 minutes each.	Rapidly provides accurate georeferenced positions of discrete points Vertical position is less accurate than other methods	Moderate to High	1:1	M	M	Keaton and DeGraff, 1996; Gili et al., 2000; Coe et al., 2003; Brückl et al., 2006

Table 7.1 Comparison of equipment and resolution for repeat ground surveys to determine surface displacement (continued)

Technique	Description	Resolution	Advantages and drawbacks	Initial Cost[1]	SCR[2]	Sites[3]	Data[4]	Reference
Terrestrial Laser Scanner (Tripod lidar)	Measures distance from instrument to millions of points on the ground surface, capable of producing detailed images or digital elevation models of the surface.	Sub decimeter to millimeter, depending on set-up, field and atmospheric conditions, instrument, and other factors.	Can monitor displacement of discrete areas on a slope face, rather than a few single points. Equipment and software are very expensive and postprocessing is complex and time-consuming.	High to Very High	1:1	M	M	Rowlands et al., 2003; Jones, 2006; Collins et al., 2007; Bowden, oral commun., 2007

1. Initial cost is in 2008 dollars and includes procurement of equipment and/or services, site work to install equipment, and obtaining initial measurement or data sample. Costs related to gaining site access are not included. Low = less than $50,000; Moderate = less than $100,000; High = less than $250,000, Very High = more than $250,000.
2. SCR stands for subsequent cost ratio and is the ratio of the costs to retrieve additional measurement to cost of initial measurement. For example, an SCR of 1:10 indicates the cost to retrieve additional readings is 1/10 of the initial cost to install and make measurement.
3. Costs for particular technique are considered to generally apply to a single site (S) or multiple sites (M).
4. Costs for particular technique are considered to generally result in data from a single measurement location (S) or multiple measurement locations (M).

Table 7.2 Aerial and satellite remote sensing methods for landslide displacement

Technique	Description	Resolution	Advantages and drawbacks	Initial Cost[1]	SCR[2]	Sites[3]	Data[4]	Reference
High-resolution aerial photogrammetry	Uses an analytical plotter or digital photogrammetry to measure displacement of photo-identifiable points on time-lapse, stereoscopic, aerial photography.	Resolution depends on quality and scale of photography and ground control. Sub-centimeter accuracy has been reported from 1:2000 scale photography.	Capable of providing accurate three-dimensional displacement. Requires ground control survey and permanent photo identifiable targets for most accurate results	Moderate	1:1	M	M	Fraser and Gruendig, 1985; Baum et al., 1998; Brückl et al., 2006
INSAR	Uses synoptic, time-lapse satellite radar images to measure landslide displacement, applicable only to show movement, and small deformations.	Millimeter vertical resolution	Applicable over large areas, but each pixel represents about 8 m to 30 m. Changes in the ground condition (e.g., plowed agricultural field) from one scene to the next prevent calculation of distance change. Provides only one-dimensional displacement, which is radial from the satellite, usually taken to be either vertical or a combination of vertical and horizontal, depending on the angle of incidence of radar beam	Moderate	1:1	M	M	Van Westen, 2004; Colesanti and Wasowski (2006) IPC04-0013

Table 7.2 Aerial and satellite remote sensing methods for landslide displacement (continued)

Technique	Description	Resolution	Advantages and drawbacks	Initial Cost[1]	SCR[2]	Sites[3]	Data[4]	Reference
Airborne (fixed wing or helicopter) Laser Scanner (lidar)	Measures distance from instrument to millions of points on the ground surface, capable of producing detailed images or digital elevation models of the surface.	Sub decimeter, depending on set-up, field and atmospheric conditions, instrument, and other factors.	Can monitor displacement of discrete or extensive areas on steep or inaccessible slopes and terrain, rather than a few single points. Permits accurate identification of changes in ground surface through obscuring vegetation or tree cover. Potential for use in conjunction with high resolution aerial photogrammetry and ground control targets to provide sub-centimeter accuracy Equipment and software are very expensive and postprocessing is complex and time-consuming.	High to Very High	1:1	M	M	Haugerud et al (2003)

1. Initial cost is in 2008 dollars and includes procurement of equipment and/or services, site work to install equipment, and obtaining initial measurement or data sample. Costs related to gaining site access are not included. Low = less than $50,000; Moderate = less than $100,000; High = less than $250,000, Very High = more than $250,000.
2. SCR stands for subsequent cost ratio and is the ratio of the costs to retrieve additional measurement to cost of initial measurement. For example, an SCR of 1:10 indicates the cost to retrieve additional readings is 1/10 of the initial cost to install and make measurement.
3. Costs for particular technique are considered to generally apply to a single site (S) or multiple sites (M).
4. Costs for particular technique are considered to generally result in data from a single measurement location (S) or multiple measurement locations (M).

Table 7.3 Techniques and sensors for continuous measurement of landslide displacement and deformation

Observation	Technique or Sensor	Description, Comments and Resolution	Advantages and Drawbacks	Initial Cost[1]	SCR[2]	Sites[3]	Data[4]	Reference
Surface or subsurface displacement	Extensometer	Can be installed across a landslide boundary or in a borehole. Resolution increases as measurement range decreases. Borehole extensometers tend to measure smaller displacement than surface displacement. 1-D, centimeter to sub millimeter accuracy	Provides highly detailed time series record of landslide movement. Cable subject to disturbance by wind and animals	Moderate to High	1:20	S	S	Corominas, et al., 2005, Ellis et al., 2007
Surface displacement	Theodolites	Motorized, computer controlled theodolites or total stations observe position of monitoring points. 3-D, sub centimeter accuracy	Accurate, non-contact measurement of selected points Requires shelter and stable base for long-term measurements	High	1:20	S	M	Angeli et al. (2000)
Surface displacement	Laser & ultrasonic sensors	Laser or ultrasonic beam aimed at target across landslide boundary. Return time determines distance, 1-D, sub-decimeter to centimeter accuracy depending on distance from target)	Limited by short range of sensors (about 15 m) and potential for landslide movement to cause misalignment between sensor and target.	Moderate	1:20	S	S	W.L. Ellis, USGS, 2007, oral commun.
Surface displacement and deformation	Terrestrial radar interfereometry	A portable synthetic aperture radar scans landslide surface at frequent intervals. Interferometric techniques reveal surface changes. Reported accuracy ranges from millimeters to centimeters.	Interferogram reveals spatial distribution of displacement Measured displacements are one-dimensional, apparatus not yet commercially available	High	1:10	S	M	Tarchi et al. 2003; Duranthon, 2004;

Table 7.3 Techniques and sensors for continuous measurement of landslide displacement and deformation (continued)

Observation	Technique or Sensor	Description, Comments and Resolution	Advantages and Drawbacks	Initial Cost[1]	SCR[2]	Sites[3]	Data[4]	Reference
Surface displacement	Wireless transceiver	Wireless networks. Other applications using wireless sensor networks are under development. (3-D, meter to sub meter accuracy)	Information available so far indicates that the spatial resolution of these networks is not yet adequate for landslide monitoring.	High	1:10	S	M	Kevin Moore, Colorado School of Mines, oral commun., 2006) (Sheth, et al., 2005)
Displacement	LVDT	The Linear Variable Differential Transformer (LVDT) consists of a mobile armature and the outer transformer windings. LVDTs are capable of high precision (2-5 X 10^{-4} mm) measurements over a small range of displacement (±5-±75 mm). Application is mainly to rock slopes where small displacements can be expected. Three-dimensional displacements are possible by using three mutually perpendicular LVDTs	Provides highly accurate measurements across cracks. Limited to use in areas of small displacement.	Moderate	1:15	S	S	Greif et al., 2004
Subsurface soil strain	Soil Strain meter	Measures soil strain over a distance of 2-5 meters, if ground is cracked, strainmeter is installed perpendicular to cracks. (1-D, displacement range of a few centimeters, capable of detecting unit strains of 0.0001)	Useful in detecting distributed strain in the soil prior to cracking or other visible evidence of deformation.	Moderate	1:20	S	S	Mikkelsen, 1996; Husaini and Ratnasamy (2001)
Surface displacement	Real-time GPS	Measurements on approximately 30-minute cycle reveal displacement at point(s) on landslide relative to base station, sub centimeter resolution, location of measurement stations can be anywhere on the landslide surface (3-D, sub-centimeter accuracy)	Capable of measuring displacement anywhere on the surface of a landslide. Can be deployed in remote locations. Requires high-bandwidth communications. Commercial applications tend to be high to very high cost	Moderate to High	1:10	S	M	LaHusen and Reid, 2000

Table 7.3 Techniques and sensors for continuous measurement of landslide displacement and deformation (continued)

Observation	Technique or Sensor	Description, Comments and Resolution	Advantages and Drawbacks	Initial Cost[1]	SCR[2]	Sites[3]	Data[4]	Reference
Tilt of ground surface or shallow borehole inclination	Tiltmeters	Repeated or continuous measurements of tilt reveal surface or shallow subsurface deformation, (1-D or 2-D, -- accuracy)	Sensitive to very small changes. Interpretation sometimes difficult without aid of displacement measurements	Moderate	1:20	S	S	Mikkelsen, 1996
Subsurface strain or displacement	Coaxial cable using time-domain reflectometry	A length of coaxial cable, grouted into a borehole, serves as the system's sensor. Electronic pulses are sent down the cable; reflected pulses are related to deformation of the cable or to pre-established reference points (crimps). Areas of offset in the resulting trace depict zones of extension or shear along the cable. Crimps, at measured intervals along the cable, partially reflect the transmitted signal and provide a more accurate scale for correlation of deformational zones to depth. Crimps appear as small negative polarity events along the trace of the waveform. Events that offset the waveform indicate deformational zones; the polarity of the offset indicates whether a zone is experiencing tensile or shear deformation. (1-D, decimeter to sub-decimeter accuracy for depth of deformation)	An alternative to inclinometers that allows for remote or continuous monitoring to establish landslide depth. Method does not permit determination of orientation of landslide movement. Instrument generally more robust than inclinometer but is destroyed or becomes ineffective after displacement exceeds a few decimeters	Moderate	1:20	S	S	(Dowding, Su, and O'Connor, 1989). Kane and Beck, 1994; Campbell Scientific, Inc., 2007

Table 7.3 Techniques and sensors for continuous measurement of landslide displacement and deformation (continued)

Observation	**Technique or Sensor**	**Description, Comments and Resolution**	**Advantages and Drawbacks**	**Initial Cost[1]**	**SCR[2]**	**Sites[3]**	**Data[4]**	**Reference**
Landslide movement at discrete depths	Borehole inclinometers	Internally grooved plastic or metal pipe grouted into vertical borehole, repeat measurements of biaxial borehole inclination at fixed depth increments reveal depth, amount, and plan-view directions of displacement. Millimeter resolution of displacement if correctly installed and carefully measured. (2-D displacement, millimeter accuracy)	Capable of providing detailed time series record of deformation at various depths Instrument destroyed or becomes ineffective after displacement exceeds a few decimeters	Moderate	1:20	S	S	Mikkelsen, 1996;
Surface or subsurface strain or displacement	Optical time-domain reflectometry	An optical fiber is used as a sensor. The optical time domain reflectometer (OTDR) measures variations in intensity of light reflections. Experimental device used in laboratory, but Baek et al. (2004) claim it could be adapted to field use. (1-D displacement)	Not commercially available	Moderate	1:10	S	S	Baek et al., 2004

1. Initial cost is in 2008 dollars and includes procurement of equipment and/or services, site work to install equipment, and obtaining initial measurement or data sample. Costs related to gaining site access are not included. Low = less than $50,000; Moderate = less than $100,000; High = less than $250,000, Very High = more than $250,000.
2. SCR stands for subsequent cost ratio and is the ratio of the costs to retrieve additional measurement to cost of initial measurement or the expected annual support cost. For example, an SCR of 1:10 indicates the cost to retrieve additional readings are 1/10 of the initial cost to install and make measurement.
3. Costs for particular technique are considered to generally apply to a single site (S) or multiple sites (M).
4. Costs for particular technique are considered to generally result in data from a single measurement location (S) or multiple measurement locations (M).

Accuracy and reliability of photogrammetric displacement measurements is depends upon image scale and quality and the availability of precise targets on the landslide surface and adjacent non-moving ground (Fraser and Gruendig, 1985) to track changes in surface displacement (Figure 7.5). Photo-identifiable points, such as boulders, shrubs, and urban features (corners, manhole covers, etc.) usually provide less precise measurements than targets. However, when using archival photography, such points are often the only basis for controlling the photography and determining displacement (Baum et al., 1998; Brückl et al., 2006). Despite these limitations, analysis of photogrammetrically derived displacements and changes in elevation has yielded information on surface geometry and depth (Baum et al., 1998; Casson et al., 2005). The development and increasing availability of digital photogrammetric equipment and software, as well as image processing software, is making these techniques more readily available; but reliable identification of points still requires operator judgment (Kääb, 2002; Brückl et al., 2006).

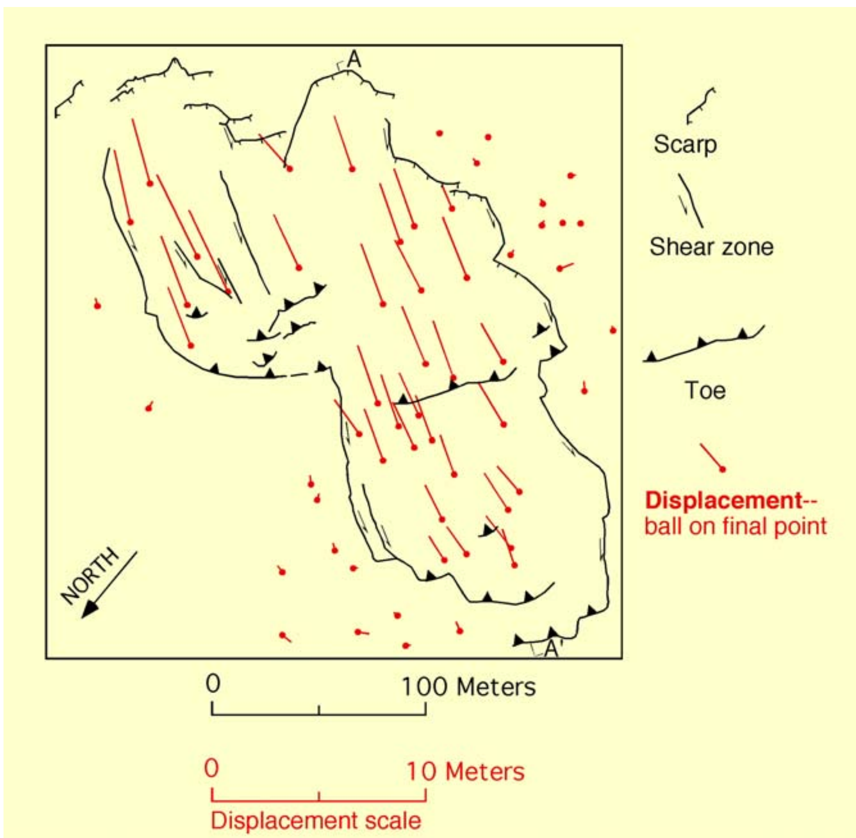


Figure 7.5 Map of landslide displacements obtained using aerial photogrammetry (Baum et al., 1998)

Satellite Synthetic Aperture Radar interferometry (InSAR) uses two satellite images taken from roughly the same point in space on different dates to determine displacement (Van Westen, 2004; Froese et al., 2005; Colesanti and Wasowski, 2006). Wavelengths of radar satellite signals are about 5-6 cm, and displacement is determined from the phase shift in the radar signal between the two measurements when compared to a reference signal.

Consequently, displacement of a fraction of one wavelength can be determined. The angle of incidence for radar signals is usually very steep, and so the observed displacement (parallel to that line of sight from the radar satellite) is mainly the vertical component except on very steep slopes. As a result of these characteristics of radar imagery from currently available radar satellites, InSAR is only capable of reliably detecting and measuring very slow vertical ground surface displacements. Vegetation and ground disruption, due to grading or landslide deformation, degrade image coherence and prevent accurate measurements (Froese et al., 2005).

Real-Time GPS

"Real-time" GPS has the advantages of being able to measure three-dimensional landslide displacement without the constraints of cables or targets required by many other forms of displacement monitoring. By installing multiple field stations on an active landslide, it is possible to monitor displacement at several critical locations on a landslide.

Terrestrial Radar

Radar interferometry implemented using ground-based instrumentation, has been tested in Europe for monitoring landslides (Tarchi et al., 2003; Luzi, et al., 2004; Duranthon, 2004; Tarchi et al., 2005). Tarchi et al. (2003) reported measuring displacement rates up to about 1 m/day with millimeter accuracy and a pixel resolution of approximately 2×2 m on the ground. Scans can be made on intervals of about 15 minutes.

Although terrestrial radar overcomes many of the challenges of satellite radar techniques, factors that degrade image coherence (quality), such as atmospheric effects, vegetation, and ground disruption, remain a challenge.

Time-Domain Reflectometry

Use of horizontal co-axial cable encased in grout and buried in the trench alongside the pipeline might provide a means to monitor differential movements and locate them using time-domain reflectometry. Time-domain reflectometry technology appears promising based on a similar application to a railway (Kane, 2007). The development of wireless networks based on low-cost sensors and wireless servers is an area of active research based on contacts with universities (Sheth et al., 2005; Moore, personal communication, 2006). However, application of these technologies to landslide monitoring awaits advances to reduce power consumption to acceptable levels for remote applications and greatly improved location accuracy of the wireless sensor technology.

7.2.2 Landslide Depth

Determination of landslide depth is critical to conducting stability analysis on existing landslides or other numerical modeling of a landslide and to planning remedial measures. Hutchinson (1983) described a number of techniques for determining or estimating landslide depth and emphasized that multiple techniques ought to be used, starting first with the more readily accessible observations. Multiple slip surfaces often exist and it is important to find the deepest. The more reliable methods depend on direct observation of offset in a borehole following landslide movement.

Borehole Probe Pipe

The simplest and least expensive of these is the borehole probe pipe, usually 25-mm-diameter semi-rigid plastic tube (commonly the riser pipe of a piezometer or observation well) that is inserted into a borehole. Metal rods of increasing length can be lowered down the tube in turn and the rod length that is unable to pass a certain point indicates the curvature of the pipe at that depth. A section of rod can be hung on a thin wire and left at the bottom of the tube; after movement has occurred, the rod can be raised to determine the lower limit of movement. The practical application of this technique in landslide investigations is to provide landslide depth information from deep piezometers that are located some distance away from the nearest inclinometer hole (Baum and Reid, 1995).

Probe Inclinometers

Probe inclinometers, first developed in the 1950's continue to be the preferred method to determine landslide depth (Mikkelsen, 1996). An inclinometer casing consists of an internally grooved, round, rigid plastic or metal pipe, which is inserted into a vertical borehole and grouted into place to provide an oriented track for the probe to travel down the boring (see Figure 7.6). The track orientation should be aligned with the fall line of the slope. Alternately some inclinometer probes use a casing of square cross section. It is important to assure the base of the inclinometer is below the landslide, or the inclinometer will simply move with the soil, and will give erroneous readings.

Repeated measurements of biaxial borehole inclination at fixed depth increments reveal depth, amount, and plan-view directions of borehole tilt, which are integrated to determine displacement. If the casing is correctly installed, probe inclinometers provide millimeter resolution of displacement amount. Landslide depth can be determined to within about one probe length, usually about 0.5 m (Figure 7.7). Several inclinometer casing diameters are available. The larger diameters will usually accommodate greater bending and thus longer life for a given amount of slope movement. Multiple installations within a landslide will provide a more complete picture of variability in amount, depth and direction of movements.

Remote measurement of landslide depth has some definite advantages and offers cost savings over regular field visits to make instrument readings. For this reason, time-domain reflectometry is gaining wider acceptance as an alternative to using probe inclinometers for determining landslide depth (Mikkelsen, 1996). A length of coaxial cable may be fastened to the outside of an inclinometer casing or other rigid tubing before installing the casing in a borehole, which is then filled with grout. As movement occurs, the coaxial cable kinks and may eventually break.

A cable tester can determine the location of cable deformation or a break in the cable (Kane and Beck, 1994). The time delay after a transmitted pulse and the reflection from a cable deformity determines its location. Unlike probe inclinometers, time-domain reflectometry can readily be automated for continuous or real-time monitoring. As with other instruments that are monitored in real time, the data collection system can be programmed to issue an alarm when critical amount of strain is detected. Landslide depths determined by TDR are comparable to those determined by an inclinometer.

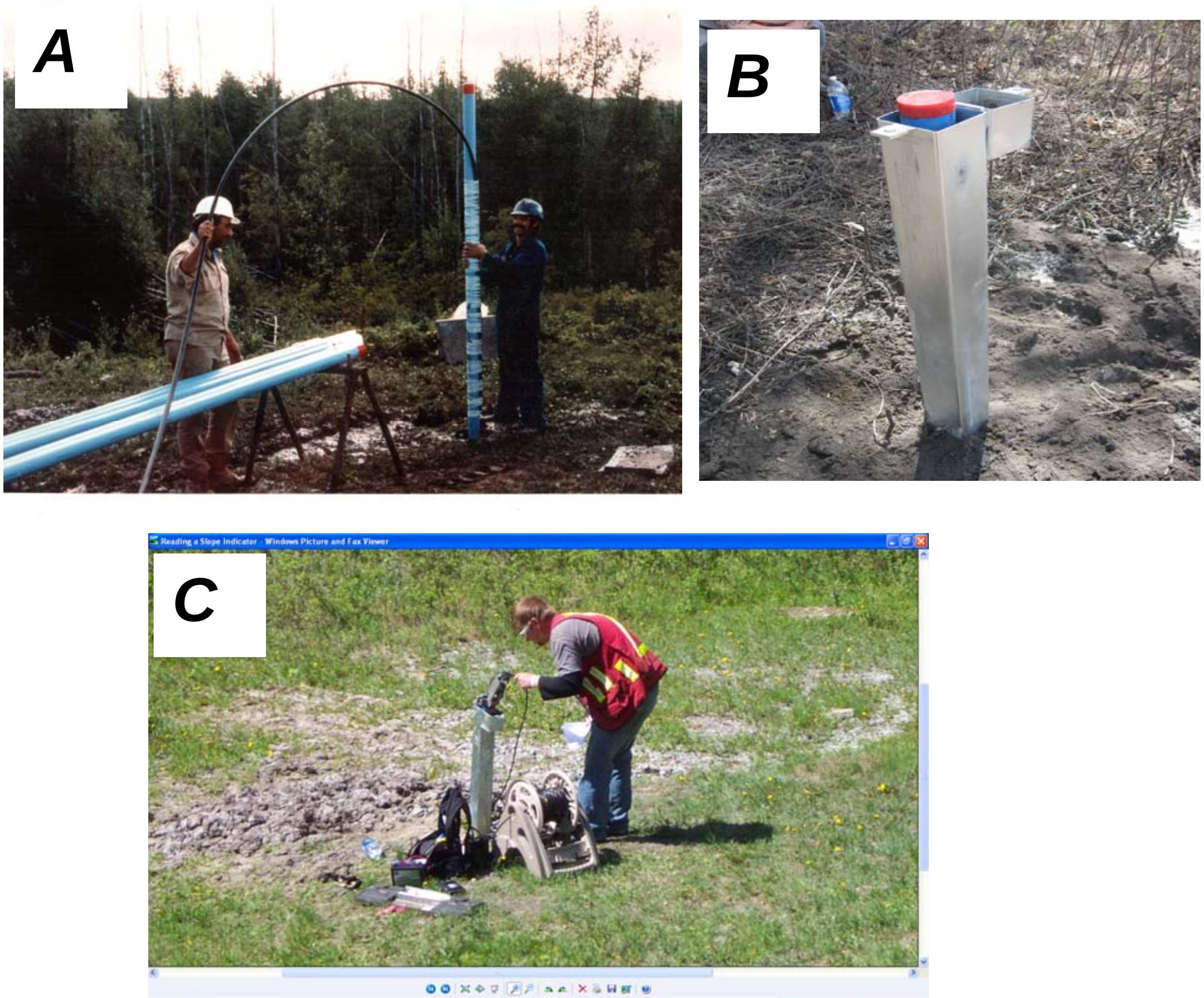


Figure 7.6 Inclinometer installation, (A) installing an inclinometer and pneumatic piezometer, (B) Completed inclinometer installation with protective metal casing, (C) Manually reading a slope indicator

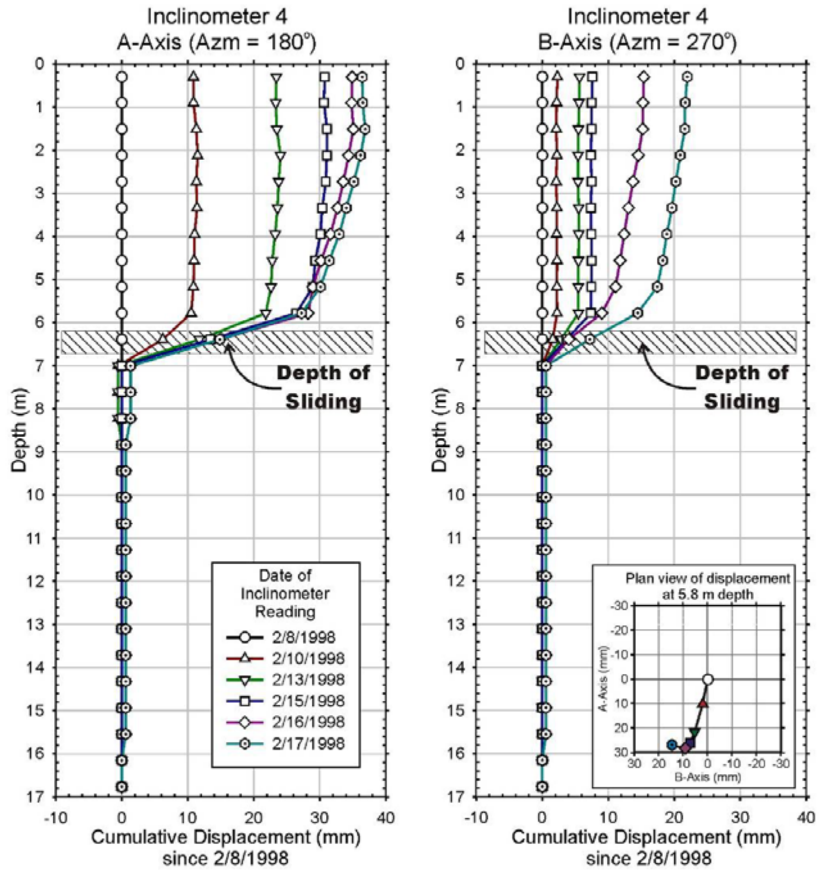


Figure 7.7 Plots of displacements versus depth at various dates obtained using a slope inclinometer

7.2.3 Pore Pressure and Water Level

Wells and open-tube piezometers have long been used to determine water table depth and pore pressure in landslides (Mikkelsen, 1996). Measurements can be made by manually probing the well or electronically by installing a pressure transducer. Pore pressure and variability at the basal slip surface is needed for slope stability analysis. In absence of more detailed data about subsurface water, water-table depth indicates the approximate pore pressure for stability analysis. The height of the column of water in an open tube piezometer or well provides an accurate indication of static or slowly changing water pressure surrounding the piezometer tip. Tthe time lag required for water levels to respond t changing pressures prevents wells and open-tube piezometers from accurately indicating transient changes in pore pressure in response to intense rainfall, earthquakes, or landslide movement. Pneumatic piezometers use a mechanical pressure transducer that consists of a chamber filled with nitrogen and a small diaphragm that senses pressure changes. Small-diameter pneumatic tubes connect the piezometer to the ground surface, where the pressure is read by a pneumatic gage. Although these gages are more accurate than open-tube piezometers (practically no time lag) due to the small amount of water displaced, they are not easily automated. Various types of electronic pressure transducers used in continuous and real-time monitoring can also be used for campaign-style monitoring.

Several types of sensors exist for observing subsurface water conditions (Table 7.4). Measurement of positive pore pressure is the most common requirement for deep-seated landslides (Mikkelsen, 1996), but measurements of negative pore pressure (suction) or soil water content may be needed in the tropics (Beneveli et al., 2004) and for shallow landslides in temperate regions (Baum et al., 2005; Tan et al., 2007). The main differences in types of instruments for measuring pore pressure concern barometric correction, response times, and ability to measure only positive pressure or a combination of positive and negative pressure. Vibrating wire piezometers are the most commonly used type in geotechnical applications, especially where long-term monitoring is planned. As their name suggests, vibrating wire sensors are based on measuring the frequency of a tensioned wire that is attached to a diaphragm. The diaphragm is in contact with the pore water; changes in water pressure move the diaphragm, which changes the frequency of the wire. Vibrating-wire sensors tend to be stable, accurate, and reliable and they can be read on intervals as short as 5 seconds. Long-term landslide monitoring usually requires readings on intervals of 10 to 60 minutes. Electrical resistance pressure transducers have rapid response times allowing multiple readings per second for dynamic applications in seismically active areas, but most commercially available models have a shorter lifespan than vibrating wire sensors.

Table 7.4 Subsurface water measurement techniques

Measurement	Technique or Sensor	Description and Comments	Advantages and Drawbacks	Initial Cost[1]	SCR[2]	Sites[3]	Data[4]	Reference
Water table depth	Water level indicator or pressure transducer in observation well	Screened or slotted casing entire depth of well allows measurement of depth of water table. Measurements can be taken manually or automatically.	Damaged instrument can be easily removed and replaced. Oversimplifies pore pressure distribution, unless used in combination with piezometers Observation well is destroyed or becomes ineffective, requiring drilling new well after displacement exceeds a few decimeters	Low to Moderate	1:10	S	S	Mikkelsen, 1996
Pore pressure	Pressure transducer in open-tube piezometer	Usually constructed from PVC pipe terminated with a porous tip. The piezometer tip is installed at the desired depth in the borehole and covered with coarse backfill. Layers of bentonite or grout above and below the piezometer tip prevent the flow of water between different horizons intersected by the borehole. Response time increases with decreasing diameter.	Damaged instrument can be easily removed and replaced. Pore pressures can be observed at depths relevant to landslide movement. Response time may be too slow to accurately indicate pore pressures associated with rainfall and snowmelt events Observation well is destroyed or becomes ineffective, requiring drilling new well after displacement exceeds a few decimeters	Low to Moderate	1:10	S	S	Lambe and Whitman, 1969; Mikkelsen, 1996

Table 7.4 Subsurface water measurement techniques (continued)

Measurement	Technique or Sensor	Description and Comments	Advantages and Drawbacks	Initial Cost[1]	SCR[2]	Sites[3]	Data[4]	Reference
Pore pressure and matric suction below depth range of tensiometers	Pressure transducer (direct burial piezometer)	Installation in cement-bentonite grout or use of high-air entry porous tip allows measurement of matric suction of soils subject to seasonal drying.	Response time faster than in open-tube piezometer, Replacement of damaged instrument requires new boring. Use of vibrating wire or equivalent piezometer transducers with electrical wire connection and readout may tolerate larger (several) decimeters displacement and permit continuous or remote readout	Low to Moderate	1:10	S	S	Mikkelsen, 1996; 2002
Matric suction	Tensiometer	Depth range is surface to about 2 m. Useful for monitoring rainfall infiltration. Conventional tensiometers require regular maintenance of fluid level, newer designs require only annual maintenance.	Rapid accurate measurements of pore pressure above water table Subject to damage by freezing.	Low to Moderate	1:10	S	S	Hillel, 1982; Baum and Reid, 1995; Baum et al., 2005
Matric suction	Thermal sensor	Experimental design, uses moisture-induced variation in thermal properties of porous ceramic tip to estimate soil matric suction	Does not require a water reservoir like tensiometers. Provides fast and accurate measurements. Not commercially available.	Low to Moderate	1:10	S	S	Beneveli et al., 2004; Tan et al., 2007
Soil volumetric water content	Time-domain reflectometry	Depth range is surface to about 2 m for profilers, certain probe designs can be buried to depths of several meters in boreholes. Relies on changes in the soil dielectric constant to observe water content.	Capable of fast accurate measurements of soil water content. For accurate measurements, must be calibrated to site soils.	Low to Moderate	1:10	S	S	Campbell Scientific, 2007,

Table 7.4 Subsurface water measurement techniques (continued)

Measurement	Technique or Sensor	Description and Comments	Advantages and Drawbacks	Initial Cost[1]	SCR[2]	Sites[3]	Data[4]	Reference
Soil volumetric water content	Soil moisture profilers and probes	Depth range is surface to about 2 m for profilers, certain probe designs can be buried to depths of several meters in boreholes. For accurate measurements, must be calibrated to site soils, relies on soil capacitance to observe water content.	Provides accurate measurements of soil water content for as many as eight different depths in a vertical profile. For accurate measurements, must be calibrated to site soils.	Low to Moderate	1:10	S	S	Baum et al., 2005

1. Initial cost is in 2008 dollars and includes procurement of equipment and/or services, site work to install equipment, and obtaining initial measurement or data sample. Costs related to gaining site access are not included. Low = less than $50,000; Moderate = less than $100,000; High = less than $250,000, Very High = more than $250,000.
2. SCR stands for subsequent cost ratio and is the ratio of the costs to retrieve additional measurement to cost of initial measurement or the expected annual support cost. For example, an SCR of 1:10 indicates the cost to retrieve additional readings are 1/10 of the initial cost to install and make a measurement.
3. Costs for particular technique are considered to generally apply to a single site (S) or multiple sites (M).
4. Costs for particular technique are considered to generally result in data from a single measurement location (S) or multiple measurement locations (M).

7.2.4 Internal Forces and Pressures

Vibrating wire earth-pressure cells and load cells have been available since the 1980s or earlier for monitoring lateral pressures in landslides or forces applied by a landslide mass on a wall or tieback systems. Most applications seem to be in monitoring dams, embankments and other earth works. Load cells have been used to monitor tension in tie-backs (Nichol and Graham, 2001) and increasing load was related to landslide movement at one end of a tie-back wall, but could not be explained at the opposite end. Practical use of earth pressure measurements in landslide monitoring and investigation appears to be quite limited at present.

7.2.5 Seismoacoustic Emissions

Fracturing of rocks or soil during the formation, reactivation, and movement of landslides emits distinctive acoustic and seismic signals that can be detected by seismic monitoring at the site. Microseismic monitoring has potential applications for detection and early warning of movement, rather than monitoring the amount of displacement. Although microseismicity has found application in other fields such as hydraulic fracturing and structural monitoring, very few applications have been made to landslides (Gomberg et al., 1996; Gaertner et al., 2000; Amitrano et al., 2007).

7.2.6 Environmental

Precipitation is the most commonly measured environmental variable in landslide investigations and monitoring. Tipping bucket rain gages are commonly used in these studies. The gages can be calibrated for millimeter or inch measurements with reported accuracy of 1 mm or 0.01 inch. Strong correlation usually exists between precipitation and pore pressure at depth, even in landslides that are many meters deep (Iversen, 2000; Lollino et al., 2006). Evidence also exists that changes in barometric pressure may affect pore pressure in low permeability clays and thereby induce landslide movement (Köhler and Schulze, 2000). Various types of recording barometers are available; however, the corrections can also be made by monitoring a pressure transducer of the same type that is used for measuring subsurface water pressures. Soil temperature can also be measured to observe the depth of soil freezing with time. This can often be related to a slowing in slope movement rates. Most pressure-transducers used in electronic piezometers and tensiometers are temperature compensated and have built in temperature sensors that can be used to make soil temperature observations. Similarly, strain gages can include temperature sensors, but these will be heavily influenced by the operating pipe temperature.

7.3 Measurement Locations

Obtaining meaningful or representative results from subsurface exploration and monitoring depends on placing boreholes and instruments in optimal locations (Table 7.5). Mapping a landslide in enough detail to identify its main features (Figure 7.8) and distinguishing shallow surficial movements from the main body of a landslide is the first step in defining those locations. Generally, landslides tend to be thinner and move more slowly near the edges than in the central part of the main body.

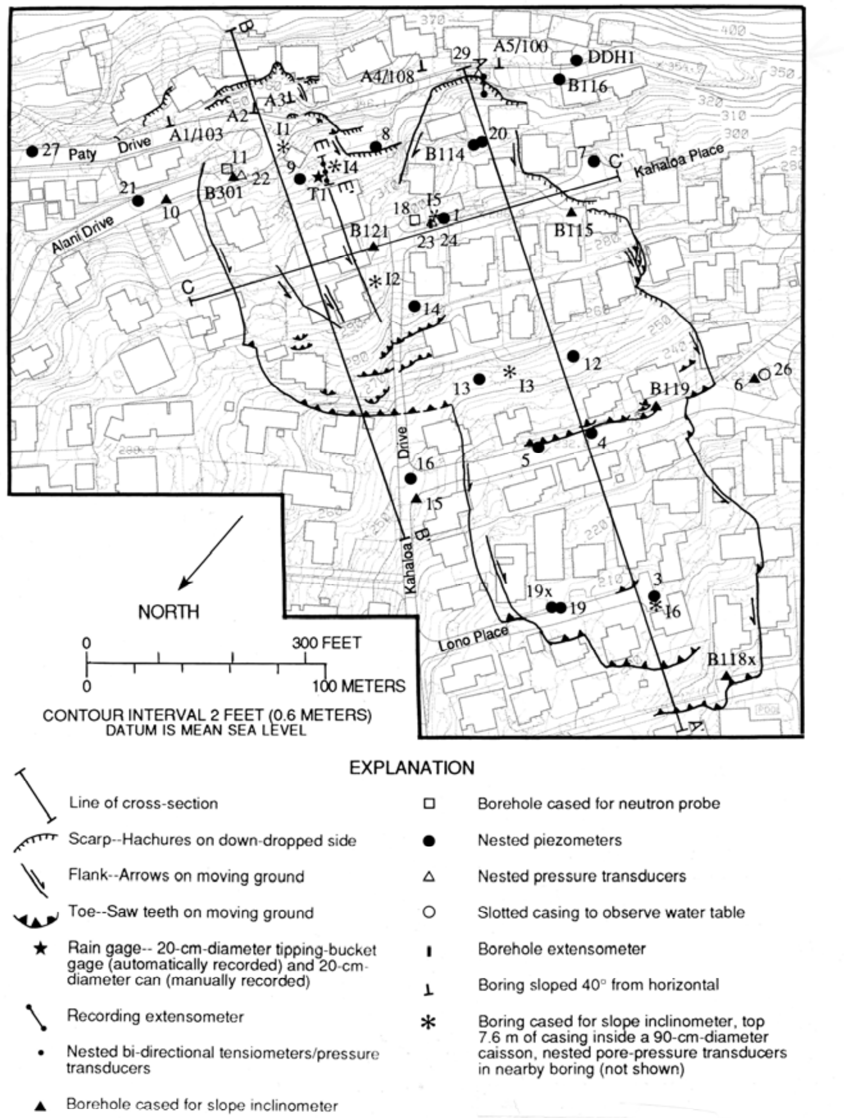


Figure 7.8 Map showing the location of boreholes and instrumentation in the Alani-Paty landslide, Honolulu, Hawaii (Baum and Reid, 1995)

The critical parameters to be observed for an existing landslide include depth and shape of the basal failure surface, pore-water pressure, shear strength, and displacement or deformation. Similar observations need to be made for potential landslides, but finding the optimal locations for these observations is much more difficult because the boundaries of a potential landslide may be difficult to define due to the lack of visible surface features. In either case, observations of the depth and shape of the basal slip surface should concentrate in the main body of the slide (or potential landslide), initially near the central axis that is parallel to the direction of downslope movement (Table 7.5). For a larger slide, the pipeline operator will want to concentrate on the area near the pipeline. It may not always be possible to locate measurement devices in their optimal locations, however, due to the restrictions of adjacent landowners, topography, or environmental restrictions. In locations with multiple pipelines and owners, it would be desirable to work with the other owners. This is not always easy and the owners may not share the same sense of priority. Optimal locations for displacement measurements are usually in the same locations as those for identifying the slide depth and pore-pressure.

Table 7.5 Preferred locations for landslide exploration and instrumentation

Observation or measurement	Primary Location	Secondary Locations	Comments
Landslide depth	Central part of main body	Upslope and downslope of central area first along main axis and then off axis	At least three points between the head and the toe to control cross-section and shape of the failure surface
Pore pressure	Central part of main body (may be multiple “main” bodies in a large slide)	Upslope and downslope of central area first along main axis and then off axis	Install piezometers as close as possible to basal slip surface. Additional piezometers above or below the basal slip surface help determine groundwater flow directions.
Shear strength	Basal slip surface	Other weak layers discovered during subsurface exploration	See earlier section on soil testing and Wu (1996) for guidance on selecting appropriate test procedures depending on soil type. Wyllie and Norrish (1996) describe procedures for determining appropriate rock strength.
Displacement and direction relative to pipeline alignment	Main body along pipeline alignment	Upslope and downslope of alignment	A line of points adjacent to the pipeline will show how displacement varies across the landslide. Additional points may be needed to detect differential movement on internal structures such as lateral shear zones.
Erosion (if present)	Watercourse bank at slope toe at pipeline crossing	Upstream and downstream of the pipeline	For landslides that occur as a result of toe erosion, often due to bank erosion by water, measuring bank loss will aid in estimating future slope movement.

Landslide measurement priorities for pipeline applications are generally driven by the need to identify conditions associated with ground displacements that might lead to pipeline damage. For application to pipelines, observations should, at a minimum, be made at several points across the landslide along the pipeline alignment (Table 7.5). Additional observations of displacement at points in the main body upslope and downslope of the alignment can help in recognizing waves of more rapid movement that are propagating either upslope or downslope towards the pipeline (Iverson, 1986). Displacement where the pipeline crosses the landslide boundaries (including internal shears) is generally the first priority, followed by displacements at points upslope and downslope of the pipeline to observe progressive or retrogressive movements that might soon affect the pipeline. Of course, the optimal locations may not be evident throughout the slide area until some data starts to come in from the first set of instrumentation. Data reviews will likely show there are locations or depths that were missed. This may trigger the need for a second field instrumentation program to hit the areas missed initially. This will be particularly applicable when investigating a potential landslide site as opposed to a site where there may be some indication of past movement.

7.4 Costs and Reliability

Available technology offers a wide range of possibilities for monitoring landslides depending on field conditions, expected style and potential consequences of movement, and the goals and budget of the monitoring program.

7.4.1 Costs

Field data collection and monitoring, whether manual or automated, are labor intensive and therefore costly activities. Design of a monitoring program for an individual landslide or a group of landslides along a pipeline corridor requires substantial field work and analysis to determine the optimal locations and types of measurements needed (Angeli et al., 2000). As a result of rapidly changing technologies, background research must be conducted to identify the most suitable components for a monitoring system. A common pitfall in this phase of landslide investigation is underestimating the true costs of instrumental monitoring programs. Estimating the cost of the initial investment in equipment and installation is relatively straightforward. However, in addition to the initial, one-time costs of background fieldwork and research, and equipment acquisition and installation, the costs of data collection, surveillance (monitoring system performance and data flow), repairs, maintenance (including replacement), and data processing continue for the life of the project. For exposed installations, monitoring equipment will need to be adequately protected against vandalism and theft. Depending on the duration of the project, the continuing costs can approach or exceed the initial investment, particularly if mobilization costs are high.

These costs should be assessed against the product of the cost of an incident and the likelihood of experiencing the incident. Incident costs should include damages (including environmental), clean-up and repair, loss of revenue, and legal.

7.4.2 Reliability

Reliability of monitoring results generally increases with the spatial distribution and density, and the frequency of measurements. Instrument accuracy and reliability aside, measurement of displacement, pore pressure, or any other quantity at a single point on a landslide has a high degree of uncertainty. Rates of movement, water levels, and other characteristics of landslides are known to be highly variable in space and time (Baum and Reid, 1995). Understanding the structure and geometry of a landslide helps ensure that measurements are made in areas that represent movement of the main parts of a landslide. Making frequent measurements at multiple locations is the key to characterizing a landslide adequately to make any forecasts about future movements. Considerable thought and analysis of all available data must go into planning of a landslide instrumentation project (Mikkelsen, 1996; Angeli et al. 2000).

A combination of technologies is generally needed to adequately monitor and characterize ground movements that potentially affect a pipeline or another linear facility. Most monitoring technologies offer adequate accuracy and precision (Tables 7.1 through 7.3). The limiting factors tend to be high cost (for some techniques), mode of deployment, and their technical characteristics. For example, wire extensometers are capable of providing detailed continuous records of movement, but they provide no directional information and their range of motion is somewhat limited. Periodic surveys (total station or GPS) are needed to provide a complete record of movement at an extensometer. Surveys to show the spatial distribution of movement are needed to detect changing patterns of movement and possible acceleration in locations where continuous monitoring was considered to be unnecessary or impractical.

The need for redundant measurements is another factor that must be emphasized. Despite efforts to engineer sensors for harsh environmental conditions, instrumental measurements routinely fail, often at critical times. A back-up system of measurements is needed to assure that measurements can continue during critical times and to maintain a continuous record. One advantage of real-time monitoring over continuous monitoring without telemetry is that instrument or measurement failure, or readings above a threshold value, can be detected soon after they occur. This allows for immediate investigation of the reason for the failure or high reading. If pipeline damage is imminent, this can provide the operator a chance to reduce pipeline pressure or shut in the line prior to an incident occurring.

The choice between automated and campaign data collection depends on economics, safety, potential incident severity, and remoteness of the landslide(s) as well as technical factors. In developing countries, where there is a nearby resident population, campaign monitoring using local labor and simple, inexpensive technology might prove effective and adequate. In areas where a pipeline crosses steep terrain and rapid movements are possible, automated monitoring may be the safest and most practical choice. However, in most areas, some combination of automated and campaign monitoring (either by site visit or remote sensing) will be needed to satisfy technical and budgetary requirements of the project. The demands of continuous or real-time data collection commonly preclude monitoring a large number of points, therefore campaign measurements and remote sensing techniques provide the spatial distribution of observations that cannot be observed by a few continuous monitoring stations alone (Coe et al, 2003).

A. SOIL SPRING DEFINITION FOR ANALYSIS OF PIPE-SOIL INTERACTION

This appendix presents equations that can be used to define non-linear springs that represent soil loading and restraint conditions on pipelines exposed to large ground displacements. Although the equations presented below reflect findings from current research into pipeline-soil interaction, the general form of the equations, and in some cases the actual values, have generally not varied significantly over the past 20 years

A.1 Unidirectional Soil Spring Equations

A.1.1 Axial Soil Springs

$$T_u = \pi D \alpha c + \pi D H \bar{\gamma} \left(\frac{1+K}{2} \right) \tan(\delta) \tag{A.1}$$

where:

- D = pipe outside diameter
- c = soil cohesion representative of the soil backfill
- H = depth to pipe centerline
- $\bar{\gamma}$ = effective unit weight of soil
- K = effective coefficient of horizontal earth pressure which may vary from the value for at rest conditions for loose soil to values as high as 2 for dense dilative soils (Wijewickreme et al., 2008)
- α = adhesion factor (Figure A.1) that is defined by an upper and lower bound

  $\alpha = 0.7\left(\frac{0.12\sigma_o}{c}\right)^{0.8} \leq 1$ lower bound

  $\alpha = 0.5\left(\frac{0.55\sigma_0}{c}\right)^{0.8} \leq 1$ upper bound

- σ_o = atmospheric pressure (100 kPa)
- δ = interface angle of friction for pipe and soil = $f\phi_m$
- ϕ_m = maximum internal friction angle of the soil
- f = coating dependent factor relating the internal friction angle of the soil to the friction angle at the soil-pipe interface

Representative values of *f* for various types of external pipe coatings are provided below.

PIPE COATING	f
Concrete	1.0
Coal Tar	0.9
Rough Steel	0.8
Smooth Steel	0.7
Fusion Bonded Epoxy	0.6
Polyethylene	0.6

Δ_t = displacement at T_u

= 3 mm for dense sand

= 5 mm for loose sand

= 8 mm for stiff clay

= 10 mm for soft clay

A significant change in the definition of axial soil springs is the definition of upper and lower bound estimates for the adhesion factor, α. In most cases, assuming an upper bound on the axial soil spring force will tend to lead to higher levels of computed pipe strain. For this reason, prior recommendations have been largely based upon enveloping the maximum adhesion factor observed from full-scale tests. However, it is possible for a pipeline configuration to experience higher strains with a lower value of α. This is most often the case when a low axial soil spring value results in high pipe line tension being transferred to a more vulnerable location (e.g., a valve station or a sharp bend) instead of to the soil.

A.1.2 Horizontal Soil Springs

Recent tests (O'Rourke et al., 2008) indicate that the horizontal soil spring relationships for dry sand are applicable to moist sand. Therefore, the following relationships adopt an approximation to the recommendations in findings from Yimsiri et al. (2004) and plotted in Figure A.2. The corresponding relationships for clay are based upon recommendations by C-CORE (2003).

$$P_u = N_{ch} c D + N_{qh} \overline{\gamma} H D \leq Q_d \qquad \text{(see section A.1.4 for definition of } Q_d\text{)} \qquad \text{(A.2)}$$

where:

N_{qh} = horizontal bearing capacity factor (0 for $\phi = 0°$)

N_{ch} = horizontal bearing capacity factor for clay (0 for $c = 0$)

$$= N^{*}_{ch} + 0.85\frac{\gamma H}{c} \leq 12$$

$$N^{*}_{ch} = 2.15 + 1.72\frac{H}{D} \leq 7.25$$

N_{qh} = horizontal bearing capacity factors for sand (0 for $\phi = 0°$)

$$= \ a + b\frac{H}{D}$$

ϕ	H/D Range	a	b	Maximum N_{qh}
35º	0.5 to 12	4	0.92	15
40°	0.5 to 6	5	1.43	23
	6 to 15	8	1.00	
45°	0.5 to 7	5	2.17	30
	7 to 15	10	1.33	

N_{qh} can be interpolated for intermediate values of ϕ between 35° and 45º and should not be taken less than 35º even if soil tests indicate lower ϕ values.

Δ_p = displacement at P_u

$$= \quad 0.04\left(H + \frac{D}{2}\right) \leq 0.10D \text{ to } 0.15D$$

A.1.3 Vertical Uplift Soil Springs

The equations for determining upward vertical soil spring forces are based upon small-scale laboratory tests and theoretical models. For this reason, the applicability of the equations is limited to relatively shallow burial depths, as expressed as the ratio of the depth to pipe centerline to the pipe diameter (*H/D*). Conditions in which the *H/D* ratio is greater than the limit provided below require case-specific geotechnical guidance on the magnitude of soil spring force and the relative displacement necessary to develop this force. The vertical uplift factor for sand is based upon an approximate fit to recommendations provided by Yimsiri et al. (2004) and plotted in Figure A.3.

$$Q_u = N_{cv} cD + N_{qv} \bar{\gamma} HD \tag{A.3}$$

where:

N_{cv} = vertical uplift factor for clay (0 for c = 0)

N_{qv} = vertical uplift factor for sand (0 for $\phi = 0°$)

N_{cv} = $2\left(\frac{H}{D}\right) \le 10$ applicable for $\left(\frac{H}{D}\right) \le 10$

N_{qv} = $\tan(0.9\phi)\left(\frac{H}{D}\right) \le N_{qh}$ (see section A1.2 for definition of N_{qh})

Δ_{qu} = displacement at Q_u

= $0.01H$ to $0.02H$ for dense to loose sands $\le 0.1D$

= $0.1H$ to $0.2H$ for stiff to soft clays $\le 0.2D$

A.1.4 Vertical Bearing Soil Springs

Vertical bearing soil springs are defined based upon the assumption that the pipeline behaves as a continuous strip footing.

$$Q_d = N_c cD + N_q \bar{\gamma} HD + N_\gamma \gamma \frac{D^2}{2} \tag{A.4}$$

where:

N_c, N_q, N_γ = bearing capacity factors (see Figure A.4)

N_c = $\cot(\phi + 0.001)\left[e^{\pi \tan(\phi + 0.001)} \tan^2\left(45 + \frac{\phi + 0.001}{2}\right) - 1\right]$

N_q = $e^{\pi \tan(\phi)} \tan^2\left(45 + \frac{\phi}{2}\right)$

N_γ = $e^{(0.18\phi - 2.5)}$ (this is a curve fit to plotted values of N_γ)

γ = total unit weight of soil

Δ_{qd} = displacement at Q_d

= $0.1D$ for granular soils

= $0.2D$ for cohesive soils

A.2 Force-Displacement Relationships for Unidirectional Soil Springs

Axial soil springs can be assumed to have a bi-linear force displacement relationship with the maximum force T_u developed at displacements Δ_t. For all other soil springs, a hyperbolic relationship between maximum soil load and relative pipe displacement can be assumed.

The general expression for the hyperbolic load-displacement relationship is provided in Equation (A.5).

$$\frac{F}{F_{\max}} = \frac{x}{Ax_u + Bx} \tag{A.5}$$

where:

F = soil spring force

F_{max} = maximum soil spring force

x = relative displacement between pipe and soil

x_u = relative displacement to achieve maximum force

Suggested values of A and B for horizontal and vertical uplift soil spring definitions are provided below:

Direction	A	B	C
Horizontal	0.15	0.85	0.46
Vertical Uplift	0.03	0.97	0.15

When the ground displacements being evaluated are much greater than the displacements to develop the maximum soil spring force, it is generally sufficient to define the force displacement relationship as bilinear with the displacement corresponding to the maximum soil spring force equal to the displacement, x_{85}, from equation (A.5) that results in 85% of the maximum soil spring force. From equation (A.5), the displacement corresponding to 85% of the maximum soil spring force can be expressed as

$x_{85} = \dfrac{0.85Ax_u}{1-0.85B} = Cx_u$ where the constant C is defined above.

A.3 Loading Rate Effects on Soil Strength Parameters

Specifying soil strength parameters for computing soil spring values must consider whether or not the rate of ground displacement results in drained or undrained soil conditions for soils with a high silt or clay content.

Based upon recommendations developed from analytical modeling (CCORE, 2003), a normalized displacement parameter, V_n, can be used as a basis for assessing whether loading is drained or undrained.

$$V_n = \frac{vD}{c_v} \tag{A.6}$$

where:

v = displacement rate

D = pipe diameter

c_v = consolidation coefficient

Undrained loading can be assumed for V_n less than 0.1 while drained loading can be assumed for V_n greater than 10. The value of c_v can be determined from oedometer tests or empirical correlations. The value of c_v is highly variable and depends upon the degree of initial consolidation and the loads generating consolidation. Considering it is reasonable to assume the soil pressures from pipe loading are much greater than the soils preconsolidation pressure, values of c_v will likely fall within a range of 1 m^2/yr to 10 m^2/yr (0.03 mm^2/s to 0.3 mm^2/sec).

Taking V_n to be 10 and adopting an upper-bound value of 10 for c_v, m^2/yr, the ground displacement rates necessary to assure undrained loading ranges from approximately 90 cm/day to 20 cm/day for pipe diameters from 0.3 m to 1.2 m, respectively.

A.4 Directional Dependency

The soil spring definitions recommended for analysis are intended to be applied independently in a soil-pipeline analytical model. This assumption is a simplification of the highly complex interaction that occurs at the pipe-soil interface.

It has long been recognized that there is some interdependence among the soil restraint acting on the pipeline. ASCE (1984) discussed the issue of axial-lateral soil spring interaction in the description of the analytical approach of Kennedy et al. (1977). Kennedy et al. (1977) determined that an increase in the maximum axial soil spring forces by factors of roughly 2 to 3 were appropriate to account for increased normal pressure on a 42-inch pipeline in moderately dense sand subject to lateral fault offsets. The axial soil spring forces in regions with relative horizontal ground displacement were equated to the product the interface friction angle, δ, and the maximum horizontal soil spring force, $N_{qh}\overline{\gamma}HD$.

Past experience with explicit finite element analyses have demonstrated that the impact of a higher axial soil loading on the pipe over a limited distance where relative lateral pipe-soil displacements occur (typically less than 50 m) has negligible effect on the computed pipeline strains. This is reasonable considering that situations where the axial soil load is an important contributor to pipeline strain, the maximum axial soil load will typically exist over hundreds of meters of pipeline, minimizing the impact of a local region with higher axial restraint. For this reason, recommendations in Honegger and Nyman (2004) did not include consideration of increased axial soil spring forces in areas of relative lateral pipe-soil displacement.

Analytical and centrifuge investigations performed by C-CORE (2008) reexamined the interaction of axial and horizontal soil forces. The C-CORE findings confirmed an increase in the same range as Kennedy et al. (1977) for the axial soil load that could be transferred to a pipeline in sand in combination with horizontal pipe displacement through the soil. More importantly, the interaction envelope recommended by C-CORE and illustrated in Figure A.5, requires that the horizontal soil spring force be reduced as a higher axial soil force is mobilized. Such reductions in horizontal soil spring force, even if they occur over a limited length of pipeline (e.g., 50 m) can significantly reduce pipeline bending strains. The interaction curve developed by C-CORE exhibited differences with other research performed on axial-horizontal interaction in sand that could not be readily explained. For this reason, these guidelines do not yet recommend that analytical methods incorporate the C-CORE sand interaction relationships. However, the potential for overestimating pipeline strains by not accounting for interaction should be recognized. Decisions on whether or not to incorporate interaction effects into an assessment of pipeline response should be made on a case by case basis, considering the acceptability of the level of conservatism associated with not accounting for interaction.

Interaction between axial and horizontal soil springs can be incorporated directly into the analytical formulation using the interaction relationships defined in Figure A.5. An alternate approach that is generally applicable with existing analysis software requires two analyses, one to identify the locations of high pipeline bending (i.e., locations of significant horizontal pipe displacement relative to the surrounding soil) and another analysis case in which the axial and horizontal soil springs are modified at the locations of high pipeline bending.

C-CORE centrifuge tests in clay demonstrated extremely low axial interaction (as noted in Figure A.1) that have yet to be fully explained. These results significantly influenced the development of axial-horizontal interaction relationships by C-CORE. Given the uncertainty regarding the C-CORE interaction relationships for clay, they are not yet considered appropriate for use in pipe-soil interaction analyses. However, the low axial soil forces in the centrifuge tests were considered in recommending a lower-bound value for the adhesion factor.

A.5 Trench Effects on Horizontal Soil Springs

In the majority of cases, soil excavated for the pipeline trench is used as backfill. In some cases, the backfill might be weaker than the surrounding soil. An example of this condition would be the use of granular backfill in a trench excavated from cemented sandstone or siltstone. It is also possible, although less common, for the backfill material placed around a pipeline to be stronger than the surrounding soil. A situation where the backfill would be stronger than the surrounding soil includes the use of compacted granular backfill or controlled density fill in trenches excavated in soft clay or peaty soils.

C-CORE (2003) presents findings from analytical and experimental investigation of the effects of backfill with lower strength than the surrounding soil. These investigations focused on clay soils and the findings generally supported an approach in which the horizontal soil springs can be defined using the strength properties of the backfill until the relative horizontal displacement between the pipeline and the soil exceeds the distance between the pipe and the trench wall. At larger relative horizontal displacements, the soil horizontal springs should be representative of the surrounding soil. This approach is illustrated schematically in Figure A.6.

Little attention has been given to the problem of stronger backfill. Conceptually, stronger backfill material should be analogous to increasing the effective pipe diameter. For extremely strong backfill, such as concrete, the increase in effective diameter will be the depth of the concrete fill. For other situations, the increase in maximum horizontal load attributable to an increase in effective pipe diameter is limited by the load at which the pipe will break out of the backfill.

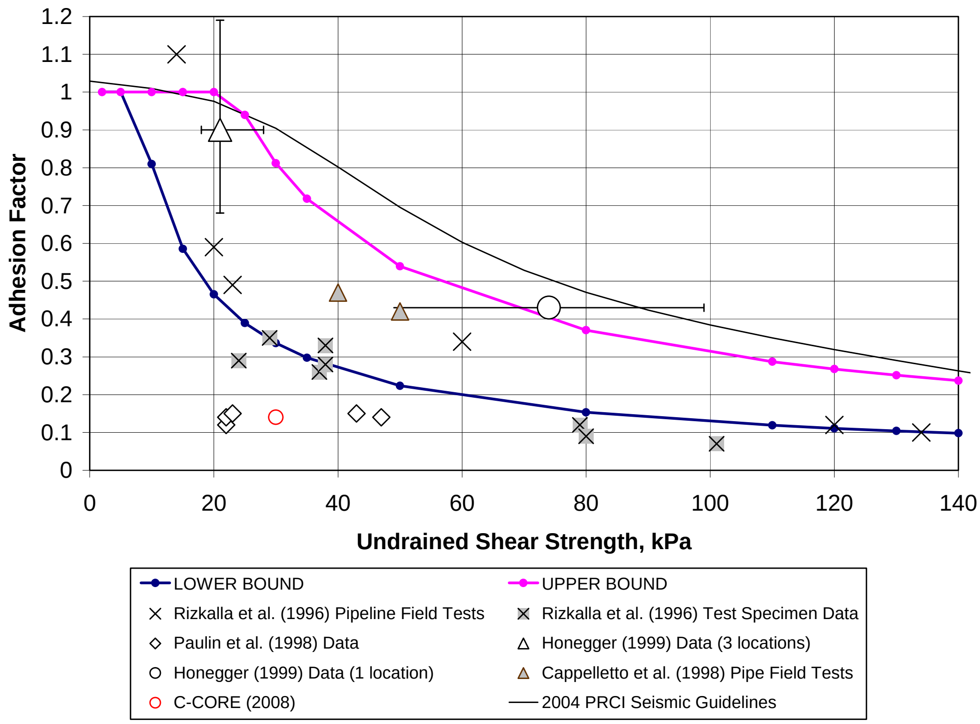


Figure A.1 Recommended bounds for adhesion factor

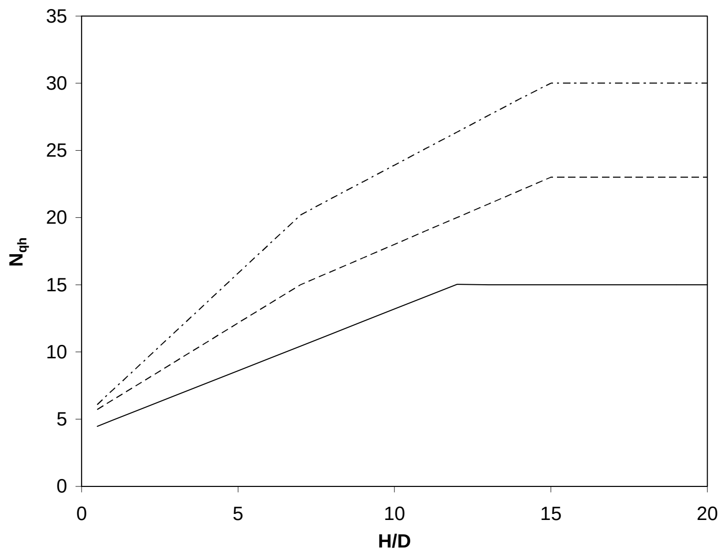


Figure A.2 Horizontal bearing capacity factors for sand

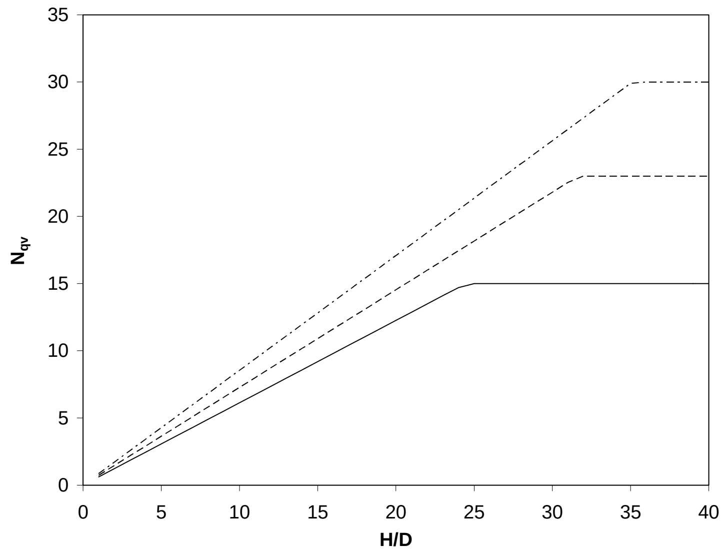


Figure A.3 Vertical uplift bearing capacity factors

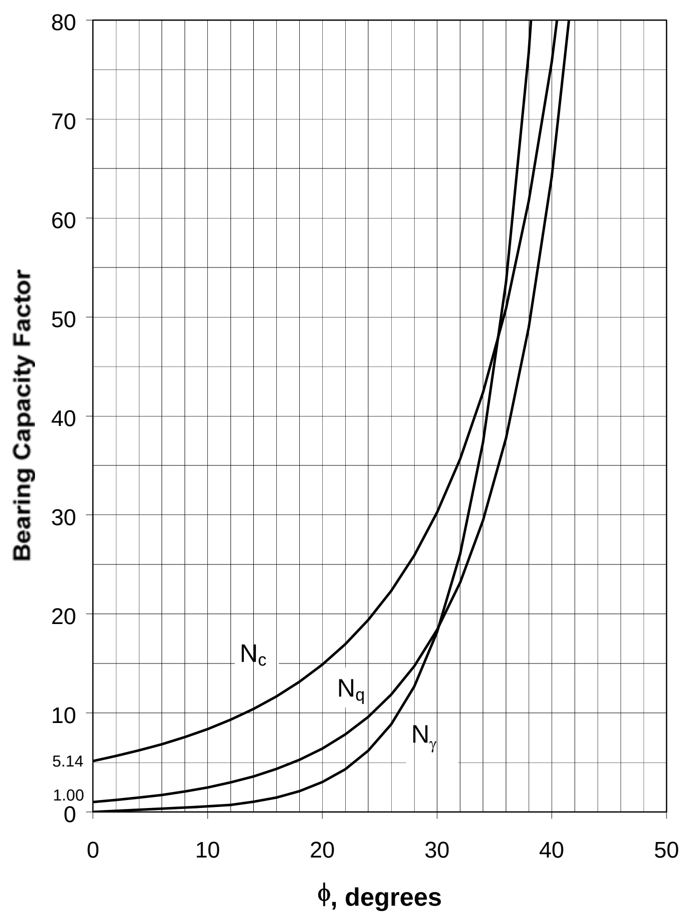


Figure A.4 Vertical downward bearing capacity factors

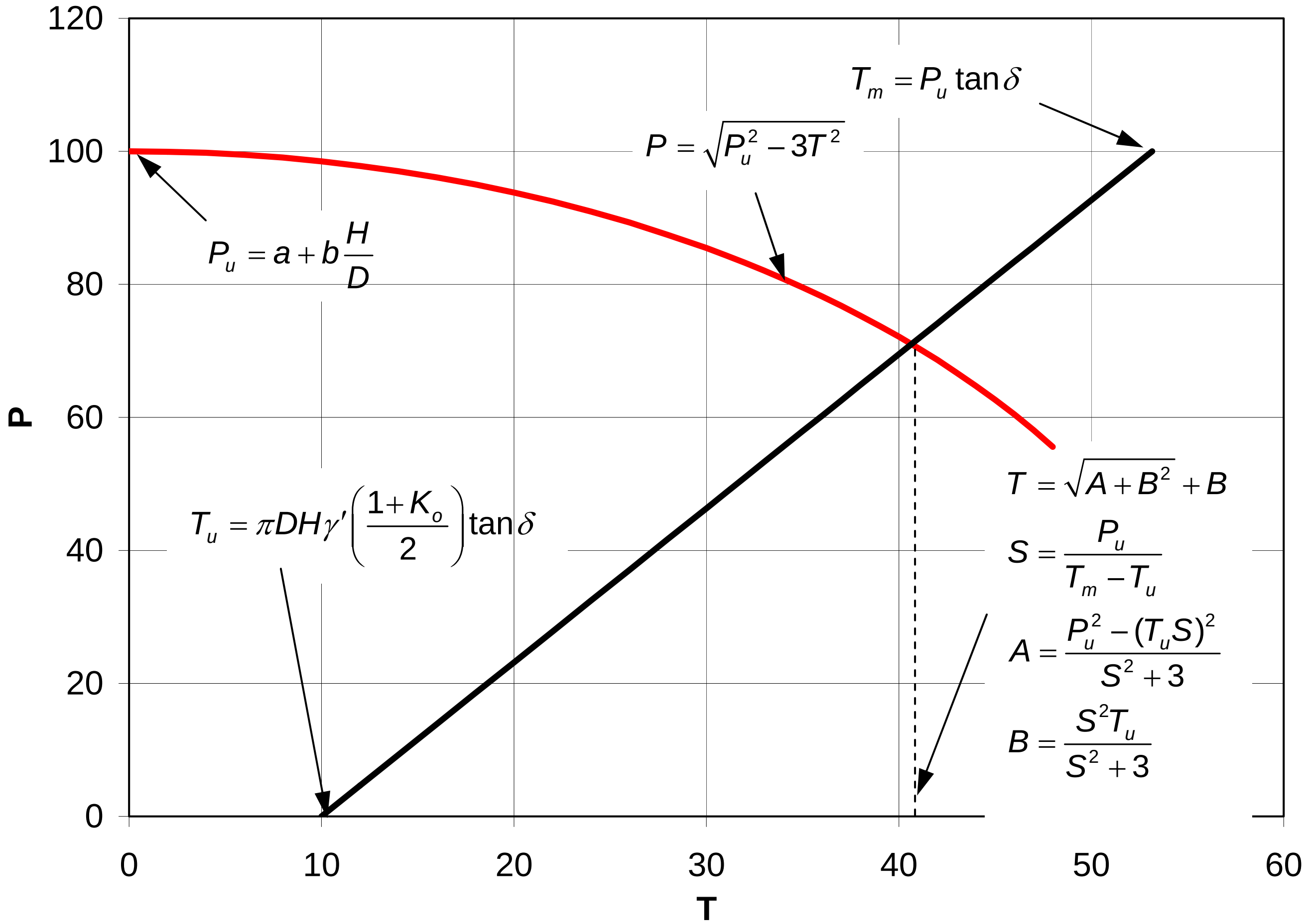


Figure A.5 Axial-horizontal sand interaction envelope

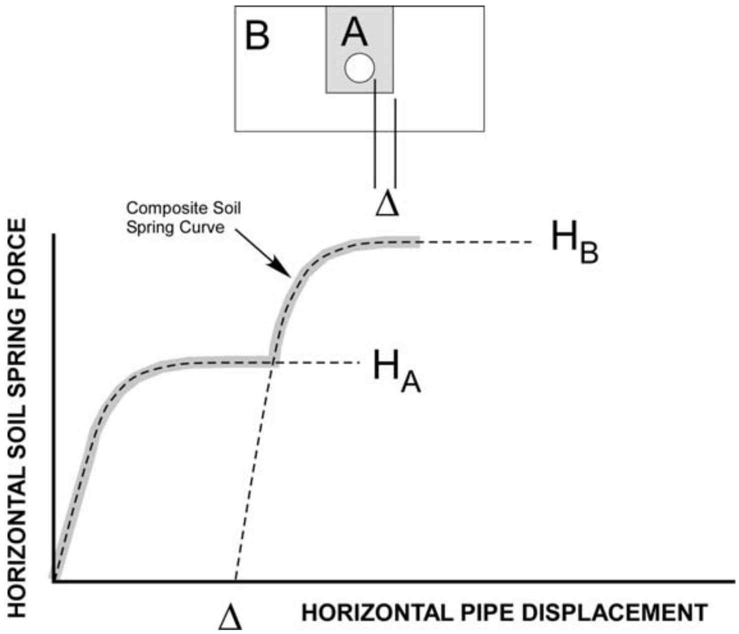


Figure A.6 Approach for defining horizontal soil springs for weak backfill

B. Algorithm for Deducing Strain From Lateral Displacements Measured with Internal Inspection Tools

Hart et al. (2008) developed an algorithm for deducing the maximum total longitudinal or axial strain from a specialized in-line inspection tool; i.e., a geometry pig, measurements of a laterally displaced pipeline. The most highly strained fibers in the pipe's cross section are the ones furthest from the neutral axis and whose total longitudinal or axial strain ε can be expressed as

$$\varepsilon = \varepsilon_e \pm \varepsilon_f \tag{B-1}$$

where:

ε_e = extensional strain

ε_f = maximum flexural strain

The maximum flexural strain , ε_f, is defined as

$$\varepsilon_f = D|\Psi|/2; \tag{B-2}$$

where:

D = pipe diameter

Ψ = change in curvature of the pipeline from its "as built" configuration

When loading in the region of the laterally displaced pipeline is predominately transverse, the axial force in this region is a constant and the inelastic extensional strain is

$$\varepsilon_e = sign(c)D|\Psi|/\pi + c \tag{B-3}$$

The constant of integration c is determined such that ε_e equals the measured extensional strain in the straight length of the pipe joint immediately adjacent to the displaced region of the pipeline; e.g., by measuring the change in length of an adjacent pipe joint from its known initial length.

The pitch θ and azimuth γ are numerically differentiated with respect to the pipe centerline distance S to develop profiles for the vertical Ψ_V and horizontal Ψ_H curvatures:

$$\Psi_V = \frac{\Delta\theta}{\Delta S} \tag{B-4}$$

$$\Psi_H = \frac{\Delta\gamma}{\Delta S} \tag{B-5}$$

from which the resultant curvature is

$$\Psi = \sqrt{\Psi_V^2 + \Psi_H^2} \tag{B-6}$$

It is important when performing the numerical differentiation that appropriate consideration be given to the gage length ΔS—too large of a gage length can result in a significant underestimate of the curvature and longitudinal strain. When two estimates for the curvature Ψ_1 and Ψ_2 are established for two different gage lengths ΔS_1 and ΔS_2, respectively, an improved estimate for the curvature is

$$\Psi = \frac{\Psi_1 \Delta S_2 - \Psi_2 \Delta S_1}{\Delta S_2 - \Delta S_1} \tag{B-7}$$

The efficacy of this technique is demonstrated in Figure B.1 in which the results from finite element simulations are compared with the improved strain estimate for ΔS_1 equal to the pig length and $\Delta S_2 = 3D$. These comparisons cover a large variation of displaced pipeline parameters, pipeline orientation relative to ground displacement, and displacement pattern applied to the pipeline. The two loading states, A and B in Figure B.1, represent different levels of applied ground displacement with state B being larger than state A. The ratio of state B to state A displacements used to simulate different ground displacement hazards varied. Full details are provided in Hart et al. (2008).

Establishing a reliable estimate for the curvature can be complicated by noise in the geometry pig signals resulting from pipeline irregularities and general signal noise. When a baseline survey of the "as built" pipeline exists, it can be used to subtract the measurement noise traceable to pipeline irregularities from the measured signal. An alternative way to filter the data is to select a gage length when computing the curvature that is greater than the characteristic wavelength of the noise; however, this type of filtering can occur at the expense of the accuracy in deducing the curvature. Therefore, care must be exercised when selecting a gage length—too large of a gage length can result in unacceptable error in deducing the curvature. In this case it may be appropriate to utilize a low-pass filter to smooth out the noise in the pitch and azimuth profiles.

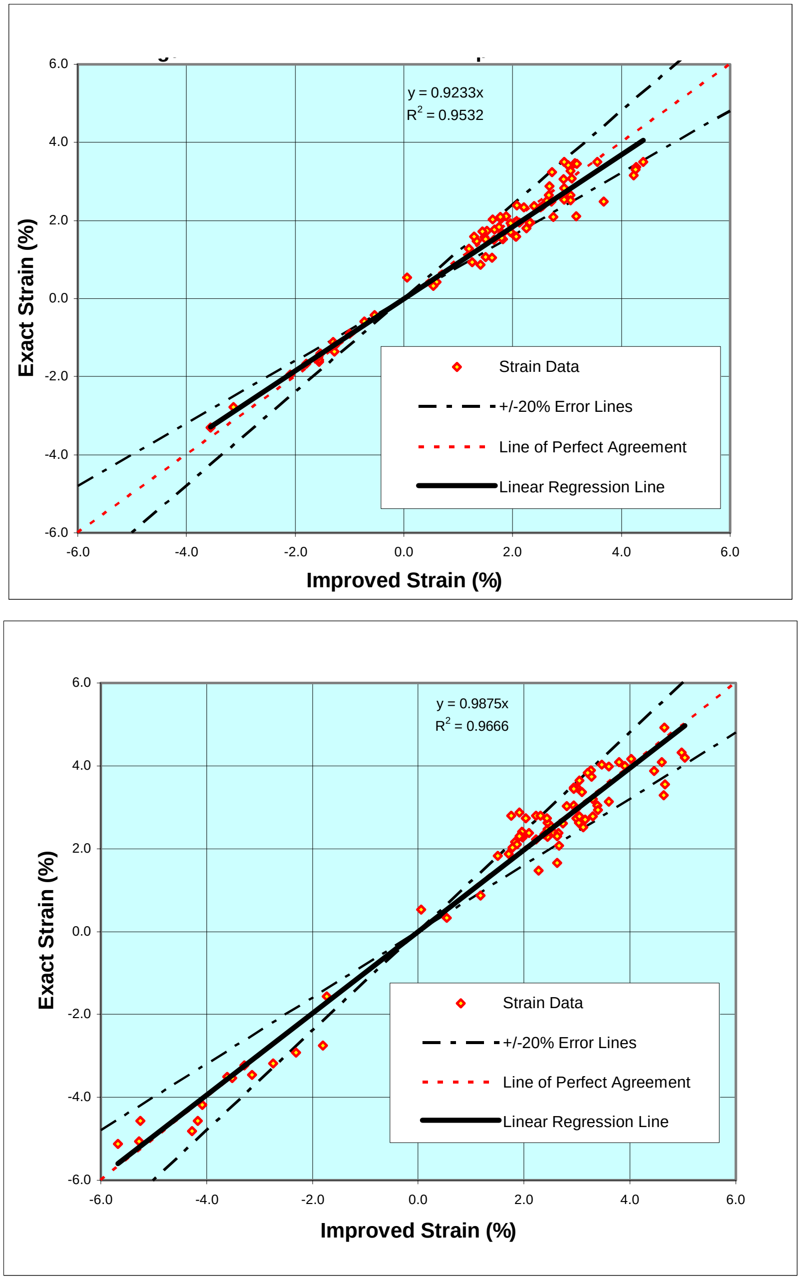


Figure B.1 Comparison of "exact" strain and strain from improved algorithm for displacement state A (top) and state B (bottom)

C. Synthetic Aperture Radar Interferometry

Synthetic aperture radar (SAR) technology, in combination with interferometry, has the ability to measure topography or ground movement. The technique is called Interferometric Synthetic Aperture Radar (InSAR), and in the context of its use in measuring relative ground movement, it is often referred to as Differential InSAR, or DInSAR. When SAR is mounted on a satellite, InSAR provides a convenient means of measuring ground movement, often without the deployment of field personnel or the expense of aircraft. Wherever vertical differential movement occurs due to subsidence, slides, settling, or creep, InSAR can often estimate the differential movement to sub-centimeter accuracy. Several radar satellites are commercially available to collect InSAR data on corridors of interest. For some locations, historical data dating back to 1992 is also available which provides a unique ability to perform historical reviews of ground movement when other data sources do not exist.

C.1 Principles of Interferometry from Space

In recent years, spaceborn repeat-pass InSAR has received much attention for its ability to generate deformation maps with unprecedented accuracy (centimeter or millimeter level). SAR is an active sensor that was developed as a means of overcoming the limitations of real aperture radars (Curlander and McDonough, 1991). SAR achieves relatively good resolution using a small radar antenna, which is an important consideration when dealing with satellites that are limited in size and are typically launched into orbits that are hundreds of miles above the Earth. To achieve this high resolution, SAR uses the motion of the radar along a flight path (or orbit) to form a 'synthetic antenna' that is much larger than its real aperture. This improves the resolution of the radar in the direction parallel to the satellite track, namely, the azimuth direction, as shown in Figure C.1. To achieve a high resolution in the across track or range direction, the radar uses a frequency modulated waveform and pulse compression to simulate a very short pulse, hence a high-resolution echo. The typical horizontal, spatial resolution obtained via current satellite SAR ranges from 8-150 m, and resolutions typically used for InSAR are 8-30 m.

Since the radar image contains the phase (ϕ) as well as the magnitude (A) of the backscattered radiation, topographic information can be derived from the difference in the phase, that is, the interferogram, between two images (Massonnet and Feigl, 1998). In particular, Figure C.1 is a simplified illustration of the variation in phase due to ground movement. The change in the distance (d) between the satellite and any point on the ground (change along the look direction of the SAR) is simply the fraction, as determined from the interferogram phase ($\phi_2-\phi_1$) for the two images, of half the radar wavelength (λ). The conversion from measured change along the look direction to the actual ground movement relies on an understanding of the ground dynamics in order to interpret the direction, and hence magnitude, of movement. When possible, measurements from another look direction may also be used to help decipher the actual ground movement.

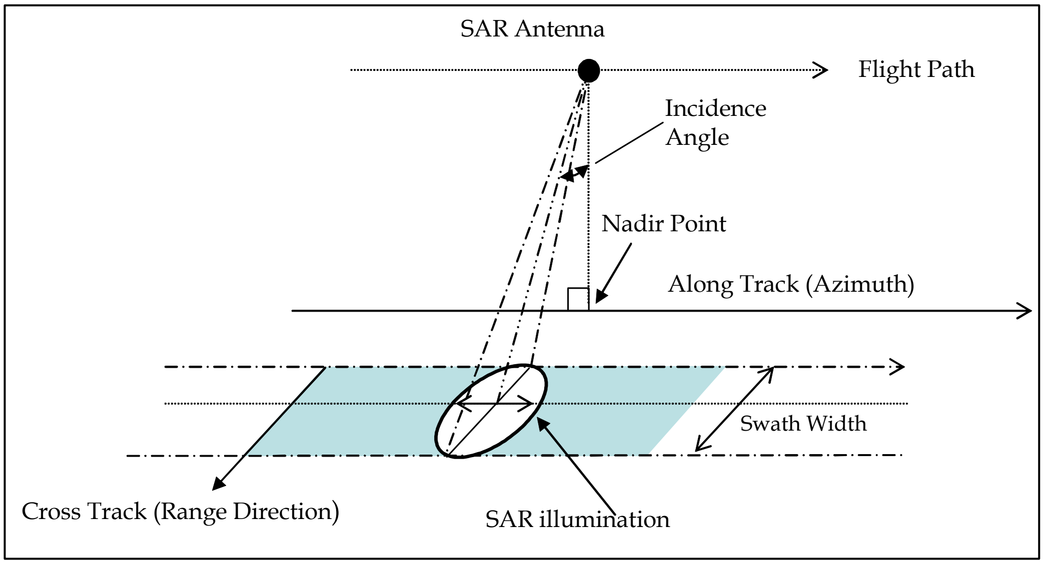


Figure C.1 Geometry of synthetic aperture radar

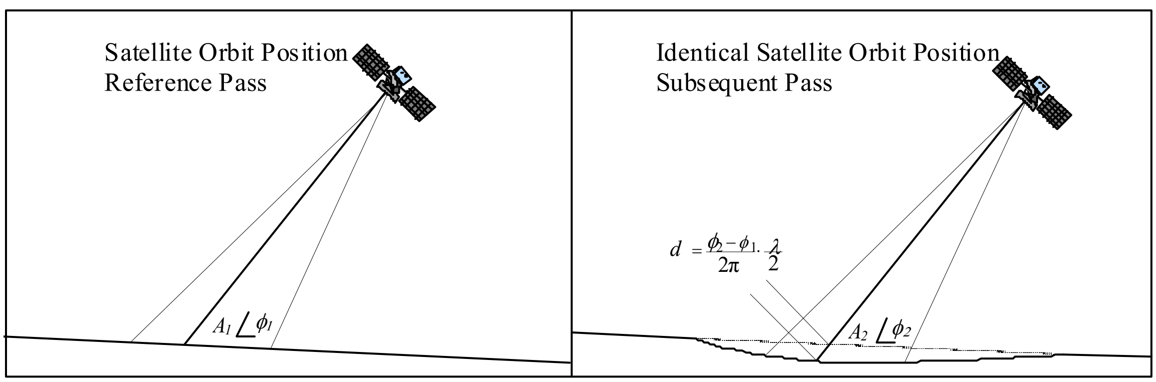


Figure C.2 InSAR measurement of ground movement

InSAR is thus based on the combination of two complex (magnitude and phase) and co-registered (aligned) radar images of the same area from an almost identical perspective. The phase difference for each pixel in the resulting interferogram is a measure of the relative change in distance between the scatterer (the ground) and the SAR antenna as shown in Figure C.3. If the observation points for the two images composing the interferogram are slightly different, a digital elevation model (DEM) can be derived from the interferogram phase, assuming that no large-scale deformation has occurred between the recordings (Zebker and Goldstein, 1986). On the other hand, deformation information can be derived if the SAR observation points are the same for the two images composing the interferogram, or if a DEM of the area is available. The latter is achieved by modeling the topographic phase contributions based on an input DEM and the geometry of the imaging. The phase contributions arising from the topography are then subtracted from the overall interferogram. This technique allows generation of very high accuracy (centimeter or millimeter level) deformation maps.

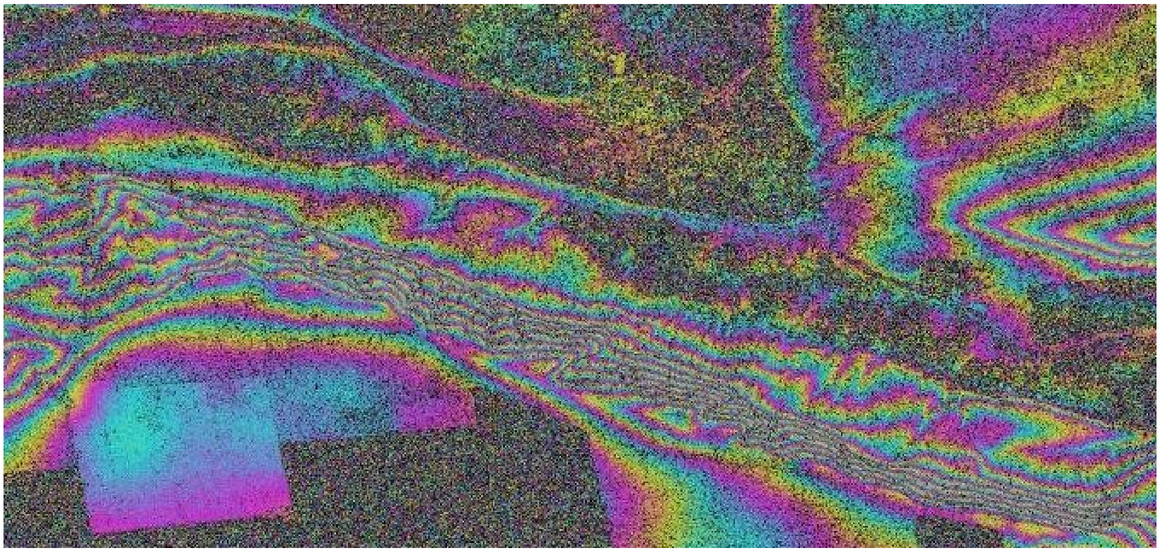


Figure C.3 InSAR interferogram (Each complete color cycle, i.e. red to red, represents 360º (2π radians) of phase shift)

InSAR is unique and hardly comparable to any conventional technique of deformation measurement. Although it is becoming more accepted, the technique has been used in a limited number of operational applications, such as volcano and earthquake monitoring as well as subsidence monitoring. Two European satellites (ENVISAT and ERS-2) and one Canadian satellite (RADARSAT-1), as well as data from previous European (ERS-1/2) and Japanese (JERS) satellites exist that are suitable for interferometric work. The spatial resolution for the SAR sensors on these satellites ranges from 30 m to 8 m – RADARSAT Fine Mode, and the orbit repeat cycles are 24 days (RADARSAT), 35 days (ENVISAT and ERS-1/2 for the majority of the mission time), and 44 days (JERS). The ERS-1/2 satellites were also operated in a tandem mode, with a 24-hour difference between the orbits of ERS-1 and ERS-2. Since there is only 24 hours between these tandem mode image acquisitions, there is generally good coherence, and hence the tandem mode is an excellent source of data for creating DEMs. The Shuttle Radar Topography Mission (SRTM) has generated DEMs at 30 m spatial resolution for areas of the Earth below 60° latitude (only 90 m resolution data have been released for areas outside the U.S.). The RADARSAT, ENVISAT, and ERS-1/2 SAR systems all use a radar wavelength of 56 mm corresponding to a frequency of 5.3 GHz, which is within the C-band radio spectrum. The JERS SAR used a wavelength of 235 mm corresponding to a frequency of 1.3 GHz, which lies within the L-band radio spectrum.

The use of satellite imagery for InSAR is convenient in that one can monitor almost any region, or as many regions, in the world as desired with equal ease. InSAR has also been successfully demonstrated from SAR equipped aircraft. This is usually more expensive but it offers the advantages of providing higher spatial resolution and the ability to control the time of data acquisitions.

The following list outlines the general methodology of InSAR analysis;

- Select and procure SAR data on the basis of meteorological data and satellite baseline.
- Extract/acquire DEM for use with the analysis.
- Perform InSAR analysis, which includes:
    - SAR image processing;
    - Image geo-referencing (to DEM and other site data);
    - Image pair registration;
    - Coherence measurement;
    - Interferogram production;
    - Phase unwrapping;
    - Phase conversion to deformation; and
    - Map product generation.
- Perform deformation analysis.
- Perform geotechnical analysis and correlation of InSAR deformation movement to in-situ data collections.

C.2 Factors Affecting InSAR Results

Because InSAR measures relative changes in phase, accuracy is on the order of fractions of a wavelength. For RADARSAT, ENVISAT and ERS satellites, one wavelength is 56 mm and measurements of ground subsidence on the millimeter scale have been demonstrated. However, the use of InSAR in the measurement of ground movement relies on accounting for any changes in the radar phase over the monitoring interval due to factors other than the change in the slant range distance. In particular, the radar phase will be affected by changes in the reflectivity (and the relative location) of the ground (temporal decorrelation), by changes in the viewing perspective (baseline decorrelation), and by changes in the atmosphere. In the worst cases, these factors will prevent the determination of ground movement from the interferogram phase. However, there are many cases where sub-centimeter and, indeed, millimeter accuracy can be achieved.

C.2.1 Temporal Decorrelation

Probably the most important limiting factor in the application of InSAR is temporal decorrelation of the ground between the interferometric acquisitions, and hence a loss of meaningful phase relation between corresponding pixels in an image pair. Temporal decorrelation usually results from changes in the complex reflection coefficient of the imaged surface (Zebker and Villasenor, 1992). Changes in the reflection coefficient are generally due to variation in the moisture content or the vegetation. Thus, decorrelation times can be as long as months to years for arid terrain and as short as several hours to several days for rainy and / or forested areas. Sparsely vegetated terrain can have decorrelation times between several days to several months. Snow-covered and frozen terrains are generally coherent over short-terms, but are sensitive to melting and snowfall. Since each pixel in a SAR image is formed by the coherent sum of the backscatter from thousands of cells on the scale of the radar wavelength, temporal decorrelation can also result from the relative movement of the scattering cells within the SAR resolution. This is particularly relevant to slope movement, since in some instances relative motion of the ground on a scale smaller than the SAR resolution may occur.

Since C-band radar has a wavelength similar to the size of small-scale vegetation characteristics — such as crop structure, foliage, and tree canopy structure — SAR images at C-band are dependent on the variations of these features, which often occur on a daily or weekly timeframe. In contrast, longer wavelength L-band radar has a wavelength on the scale of tree trunk and branch structures, which generally change over a much longer timeframe. Thus, in vegetated areas, the longer wavelength SAR provides the possibility of obtaining useable interferometric pairs over longer timeframes than provided by C-band SAR.

The problem of temporal decorrelation due to changes in the complex reflectivity of the ground or the vegetation can be mitigated through the use of phase-stable targets, such as buildings, other anthropogenic infrastructure, rock or gravel outcroppings, or radar reflectors — as shown in Figure C.4 — that are installed specifically for this purpose. In these cases, however, the ground movement is measured at isolated points, and only if the spatial density of such points is high, can a continuous spatial estimate be made of the ground movement.

Figure C.4 Reflectors can be used to mitigate the problem of temporal decorrelation

Radar reflectors may be either passive or active — the former most often being constructed from metal panels as shown in Figure C.4, and the latter being constructed with receive and transmit antennas linked through an amplifier. Active reflectors are smaller but they require a power source and are generally more expensive. Passive reflectors come in several variations, including dielectric lens, flat panels (mirror-type), dihedrals (two perpendicular panels), and trihedrals (three-panel corner, as in Figure C.4).

In recent years, interest has been increasing in the use of permanent scatterers for SAR interferometry (Ferretti, et al., 2000, 2001, Werner, et. Al., 2003). It is based on identifying point targets that are coherent over an extended timeframe. By measuring the interferometric phase at such points over multiple timeframes, the topographic, atmospheric, and decorrelation noise contributions can be isolated, thereby permitting an accurate assessment of the differential phase due to ground movement. Specifically, the technique relies on using the characteristic temporal and spatial scales of these contributions to aid in their identification. Accuracies approaching a millimeter have been obtained based on interferogram stacks of 40 to 60 ERS-1/2 scenes.

C.2.2 Baseline Decorrelation

Variation in the phase occurs with different viewing geometries, since the relative locations of the scattering cells depend on the viewing position (Zebker and Goldstein, 1986). The different viewing geometries are denoted by the satellite baseline, or the difference in orbit position from one satellite pass to the next. Satellite baseline position (both parallel and perpendicular) is illustrated in Figure C.5. The variation in phase due to baseline is beyond the simple distance and phase relationship that is the basis of DEM and deformation measurements. The variation of phase with viewing geometry leads to a maximum separation between two observation locations that can be used for InSAR analysis. This maximum separation is called the critical baseline, and is dependent on the radar wavelength, the sensor-target distance, the range resolution and the incidence angle (the angle of the satellite look direction from nadir, i.e., perpendicular to the ground). Further, the coherence of an interferometric pair depends on the spectral correlation between the two observations at different viewing geometries (Gatelli, et al., 1994).

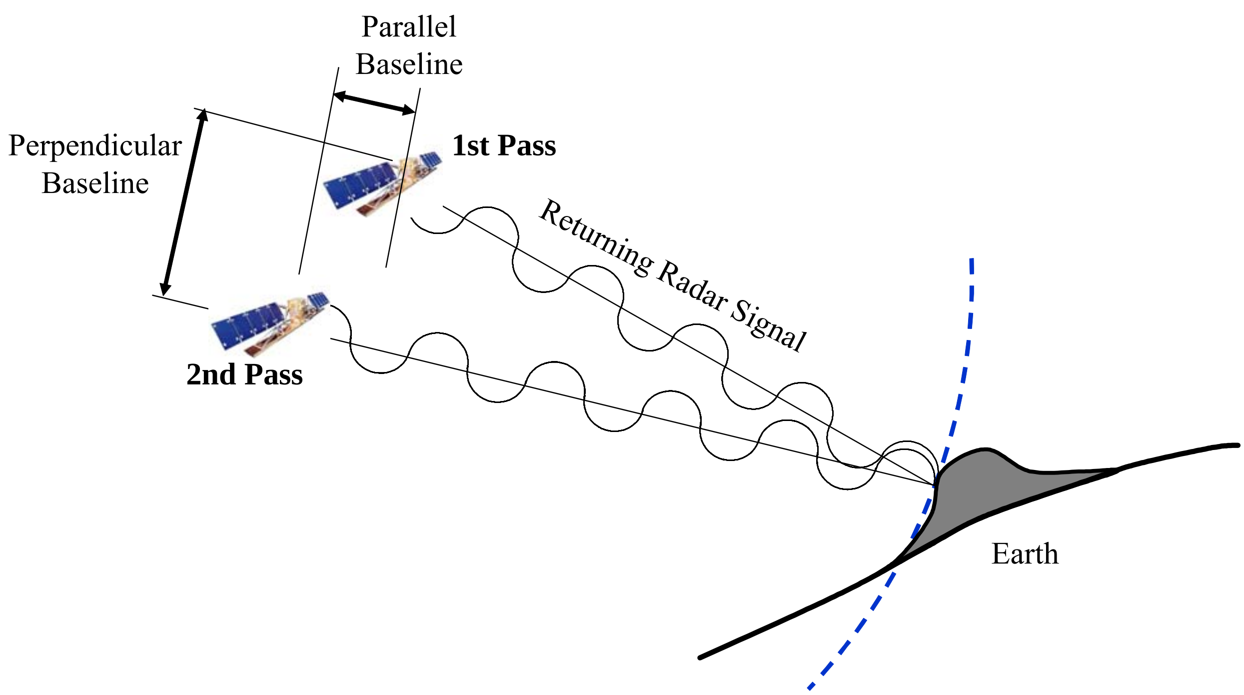


Figure C.5 Orbit baseline changes can produce varying phase shifts

One should note that when a point target dominates the radar return within a SAR resolution cell, there is no baseline decorrelation. This, of course, assumes that the radar response from the point target is isotropic, at least within the variation of the SAR viewing geometries. The use of point targets, therefore, has the advantage that it is not sensitive to orbit baseline separation, so that it permits the use of available SAR images with larger baselines, often enabling more frequent monitoring.

C.2.3 Atmospheric Effects

There are numerous studies of the influence on InSAR of atmospheric effects, ranging from homogeneous effects to heterogeneities in both the troposphere and the ionosphere (Tarayre and Massonnet, 1996). Phase shifts due to homogeneous atmospheres produce additional interferometric fringes and can be accounted for by adjusting the satellite baseline. Given sufficient coherence, heterogeneities can often be recognized on the interferogram. Alternatively, the variation due to atmospheric effects can be isolated from multiple interferograms. (Fruneau and Sarti, 2000) This is also the approach in using interferometric stacks and in permanent scatterers analysis. In particular, for large numbers of interferograms, the atmospheric effects can be identified as a random process over time and thereby separated from other contributions to the interferometric phase.

C.3 Slope Movement Monitoring

The use of InSAR to measure ground movement along slopes is not as common as other applications, such as measuring crustal deformation due to earthquakes and volcanoes, and measuring subsidence, especially in urban areas. There are issues associated with using InSAR that are accentuated when it is applied to measuring slope movement. This includes the sensitivity of the SAR system to the actual slope movement, based on its look-direction and spatial and temporal resolutions.

C.3.1 Look Direction

For the current polar-orbiting SAR satellites, the look direction (except at high latitudes) is generally either east or west, for either ascending or descending orbits respectively, as shown in Figure C.6. These SAR systems are, therefore, sensitive to movement along slopes facing either east or west, and insensitive to movements in either a north or south direction. Furthermore, if the SAR look direction faces the slope, then once again the SAR is not very sensitive to movement along the slope, and, in addition, the slope face may be imaged at close to the same SAR slant range, as seen in Figure C.7. This effect is worst when the slope inclination is equal to the SAR incidence angle. For steeper slopes, the SAR image suffers from layover, since the upper section of the slope is closer to the sensor and therefore it appears to be laid over the lower section.

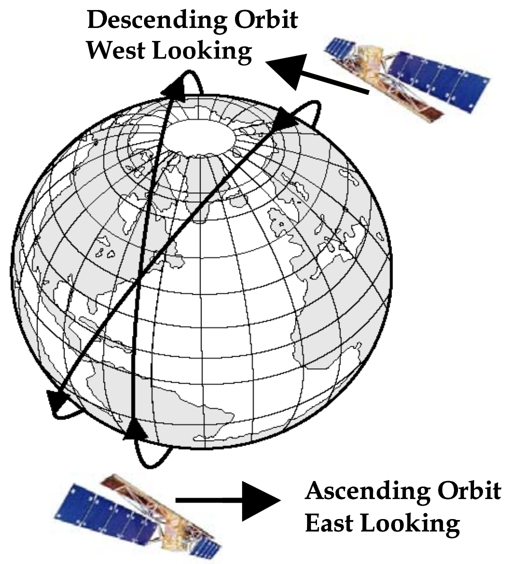


Figure C.6 Polar orbiting satellites have an east-looking and west-looking perspective (RADARSAT International, 1995)

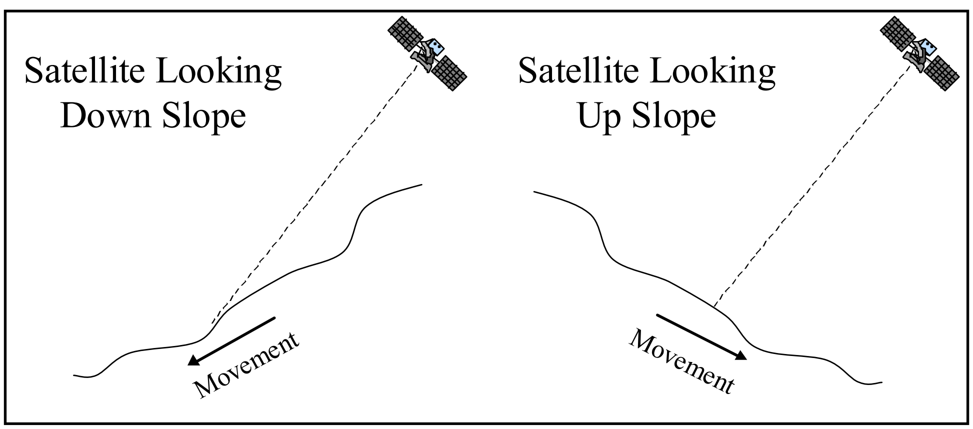


Figure C.7 Example of satellite looking up-slope and down-slope

Movement along specific slopes is usually defined by characteristic spatial and temporal scales. These may or may not be congruous with the SAR spatial and temporal scales. In particular, small and / or fast moving slopes are difficult to measure using spaceborn InSAR, since the spatial resolutions of the available sensors at present are 8 m to 30 m, while the orbit repeat cycles are 24 days for RADARSAT and 35 days for ENVISAT and ERS-2. If movement along larger slopes is composed of different mechanisms acting on smaller blocks, then once again the spatial resolution of the SAR may be a limiting factor in identifying these mechanisms.

In instances where a slope has to be monitored at high spatial and temporal scales, ground-based SAR systems have been used. Such systems have been used, for example, to monitor landslides in Valdarno, Italy (Pieraccini, et al., 2003) and in Schwaz, Austria (Leva, et al., 2003). An additional advantage of employing this system for high frequency monitoring is that the temporal decorrelation is minimal over the short timeframe between acquisitions.

C.3.2 SAR Layover and Shadow

In addition to considering issues of coherence, baseline and atmosphere, slope monitoring with SAR must also consider the slope direction and steepness along with the SAR incident angle and look direction. During the SAR acquisition, radar shadow will occur whenever the radar is looking downslope and the radar incidence angle is greater than the slope angle. In this case, the area obscured by the top of the slope will obviously not be imaged. Conversely, if the radar is looking at the slope and the radar incidence angle is less than the slope angle, then the top of the slope will be imaged before, or laid over, the lower part. In areas of either layover or shadow, the particular SAR acquisition geometry cannot provide information on slope movement. ERS-1/2 has a fixed incidence angle of approximately 23°, which is considered to be steep. RADARSAT on the other hand has variable incidence angles. For the highest resolution imagery from RADARSAT (i.e. Fine Mode), the incidence angles vary from 36° - 48°. Further explanation of this effect is provided below.

When a spaceborn SAR looks down and to the side toward a steep mountain, many objects on the mountain's facing slope may appear to be located at the same distance from the satellite. Since those many objects are located at nearly the same distance from the SAR, their backscattered signals will return to the spacecraft at about the same time. The SAR sensor will interpret this as a single object located at that distance; consequently the SAR image will be very bright at that location, in which all those responses from the separate objects are mapped into one location. This is called foreshortening in the case with the objects' distances are closely spaced, or layover in the extreme case where responses from, say, a mountain's peak are positioned before surrounding locations. Figure C.8 below shows an illustrative example of this for one particular incidence angle. In this case, the entire left side of the mountain cannot be imaged properly by the SAR.

SAR illumination is much like solar illumination, and thus shadowing will also occur in cases where the front side of a slope or mountain creates a shade effect on the back side of the slope or mountain. An example of this is shown in Figure C.9, except in this case, a much shallower incidence angle is used. After obtaining the very response from the front side of the slope, the SAR will suddenly sense very little or no response from the mountain's opposing face. Note that the mountain's back facing slope may be nearly parallel to the incoming radar, making it seem to the SAR that there are few responses for a significant distance.

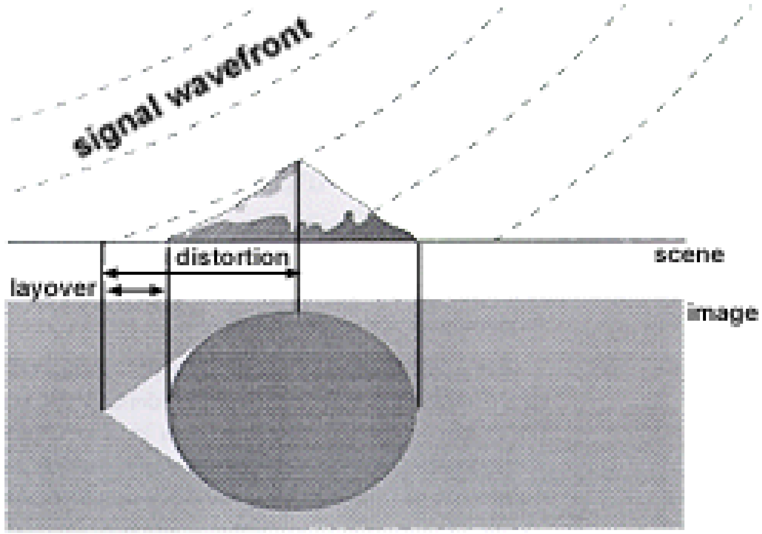


Figure C.8 The concept of layover in SAR image acquisition

As you can see in Figure C.9, shadow is much worse for shallow incidence angles than for steep ones. In Figure C.8, there was almost no shadow on the right side of the slope, but in Figure C.9, the entire left side of the slope is shadowed. So, for example, ERS will have less shadow problems than RADARSAT-1. However, as Figure C.9 shows, the satellite does not have a problem imaging the left side of the slope, as did the satellite in Figure C.8. This implies that there is a trade off; satellites with shallow incidence angles will have a more difficult time imaging all slopes of an area of high relief if there are regions of shadow. However, shallow incidence angles may be more suitable for imaging certain portions of some steep slopes, depending on the geometry of the slope.

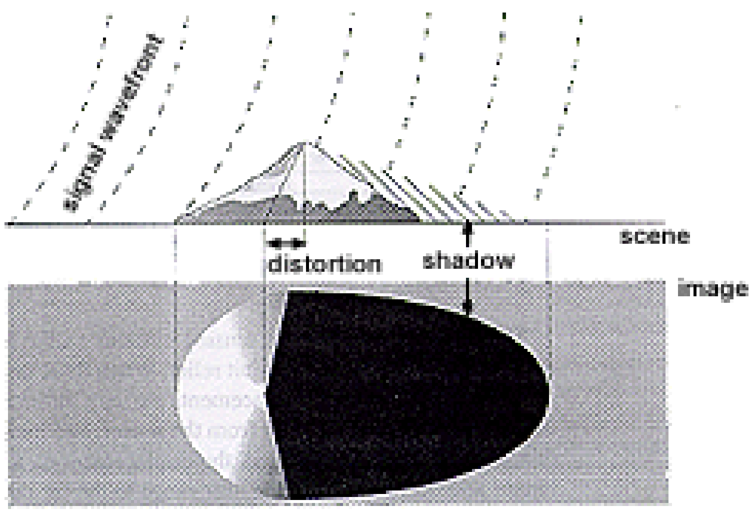


Figure C. 9 The concept of shadow in SAR image acquisition

C.4 Geo-referencing and Control of SAR Images

The native format of a SAR image and the resulting movement data derived by InSAR is a raster image of data points on a uniform grid pattern. These data are not unlike that of aerial photographs, however, points on the ground are representative of microwave radiation echoes (or interpreted ground movement) rather than solar illumination. As a consequence, SAR data can be placed on a ground coordinate system (i.e., geo-referenced) using methodologies already established for use in aerial or satellite photogrammetry. These methodologies usually involve the use of surveyed ground control points (GCPs) located in the region of interest that can fix a point in the image to a location on the ground.

C.4.1 Collection of GCPs for Geo-referencing of SAR Images

For aerial photography, usable GCPs are objects or monuments that can be easily identified and surveyed in the air photo such as the corners of buildings or road intersections. In the absence of easily identifiable GCPs (e.g., in a rural area), control points can be placed throughout a region of interest prior to image acquisition. For example, a large white cross or square placed on bare ground can serve as a convenient and inexpensive benchmark; this artificial monument can be surveyed and subsequently removed after the air photo is captured. A suitable number of these GCPs located (or placed) throughout the air photo will allow the image to be tied to ground coordinates (e.g., state plane) and subsequently projected to a particular map projection (e.g., Universal Transverse Mercator, (UTM)).

Since SAR images are comprised of microwave echoes, the method of collecting GCPs is slightly different than that performed for air photos. Many objects that are highly visible in air photos are not visible to the SAR instrument. For example, painted white lines on a road are highly visible in an air photo but are invisible to the SAR. Therefore, GCPs must be selected that are highly visible to the radar and are easily geo-located or surveyed. For example, roadways are generally visible in SAR images and road intersections can be used as GCPs. Suitable natural GCPs include lakes and river edges (that generally are dark in SAR images) and ridgelines (that generally are bright in SAR images). Corner reflectors as shown previously in Figure C.4 are most commonly used as artificial GCPs in SAR because they show up very brightly as point targets in the SAR image. They can also be pegged in place permanently if necessary and are easily surveyed with traditional equipment.

Rigorous geo-referencing of SAR images is particularly important for the application of InSAR. Raw SAR data received from image vendors is typically poorly geo-referenced with geo-referencing errors on the order of hundreds or thousands of metres. New generation satellites such as ENVISAT and RADARSAT-2 have much better base geo-referencing due to the availability of onboard Global Positioning System (GPS) for precision orbit estimation. This does not eliminate the importance of a rigorous manual geo-referencing procedure using GCPs from the most accurate source available. Manual geo-referencing allows a more accurate placement of the SAR image (and the interpreted InSAR derived movement data) in a coordinate system that is common with other forms of data, such as GIS layers (road networks, infrastructure, etc.), elevation models and topographic maps. This will facilitate a more accurate assessment of the implications of movement measurements with InSAR. In addition, InSAR requires the alignment of the SAR images with a digital elevation model to remove topographic phase; thus the precise alignment of SAR images to a reference coordinate system common with the elevation model is important. Otherwise, residual topographic phase might remain in the InSAR derived movement image, and in the extreme case, might mask actual movement data which would lead to incorrect movement interpretations.

C.4.2 Sources of SAR GCPs

As mentioned above, surveyed corner reflectors are one of the best sources of GCPs for SAR data. While this is the case, it is not always possible or even necessary to place corner reflectors in the region of interest. Cost of procurement and placement of reflectors may preclude their use in a project and a new SAR image must be acquired after reflector placement to reference previously acquired SAR images in data archives. Often, there are other equally suitable data available for geo-referencing purposes, which involve other raster or vector data that have been previously referenced using survey and control methods. These data include topographic maps, orthophotography, and photogrammetry, and are described in the following subsections.

C.4.3 Topographic Maps

Topographic maps provide the most comprehensive coverage as a control source. Thus, this is the most readily available source of SAR GCPs. In general these are available for all areas at a scale of 1:50,000. These can be used to geo-reference SAR data to within 20 metres horizontal accuracy. Features such as road intersections, water body edges and ridgelines are easily identified on these maps and their corresponding geo-locations can be used as GCPs for the SAR image. For many InSAR projects, the geo-spatial accuracy obtainable using topographic maps is often more than suitable.

While these topographical maps are readily available, they are limited by the geo-spatial accuracy of the base map and the limited quantity of natural or manmade GCPs that may be available throughout the image. In the case of flat rural terrain with few roads or other infrastructure, it can sometimes be challenging to find more than a couple of suitable GCPs. The use of topographic maps might also result in inaccurate geo-referencing of the SAR images if the information on the topographic map is not up-to-date. For instance, recent road re-alignments may not be reflected in the topographic map, and this could lead to incorrect placement of the SAR image if the new road alignment was used as a source of GCPs. This is particularly relevant in this project; two of the three sites used in this project have had extensive road work performed within the last eight years.

Another convenient source of SAR GCPs is aerial photography that has been properly geo-referenced to a standard datum and orthorectified to a suitable map projection. These include orthophotography and site specific photogrammetry.

C.4.4 High Resolution Orthophotography

Existing digital orthophotography can provide highly accurate horizontal control, assuming availability in the study areas. Many counties or local consortiums maintain high resolution orthophotography as part of their electronic Geographic Information System (GIS). Pixel resolutions typically range from 15 – 60 cm. Since this is a 2-dimensional product, only horizontal control can be obtained. Vertical control could conceivably be obtained (interpolated) from the underlying DEM. Although these are not as accurate as a contour DTM, elevations obtained are certainly suitable for the desired application. Elevation inaccuracies could range up to 3 m.

C.4.5 Photogrammetry

Existing photogrammetry projects are another source of controlled SAR GCPs. The accuracy of photo identifiable control is relative to the flying height of the photography, which can vary widely depending on the mapping requirements (i.e. map scale and contour interval). Typical photogrammetry mapping projects will range from 1:600 to 1:2400 in map scale and will yield horizontal accuracies ranging from 30 – 150 cm. Vertical accuracies of the underlying elevation model, assuming contour intervals from 30 – 150 cm, will range from ± 15 – 75 cm.

C.4.6 Summary of GCP Collection

In the case of the examples provided above, GCPs are only available within the extent of the established air-photo and associated control. In cases where the high-resolution air photo coverage is much smaller than that of the SAR image, additional GCPs must be collected from other sources.

It is important to note that the presence of comprehensive site survey and control data will not alone facilitate the geo-referencing of SAR images unless the benchmarks used in the survey can be visualized in the SAR image. This is most often not the case, since the monuments used for surveying (i.e., pegs or rebar rods) are not visible in a SAR image. The site survey and control information is only useful if it is tied in with a source of usable GCPs, which is most often an orthorectified air photo.

C.5 Site and InSAR Suitability

There are several factors to be considered when determining a site's suitability for InSAR monitoring. These include;

<u>Slope Alignment:</u> Slopes that are ideal for InSAR monitoring are those facing in a general East or West direction. This maximizes sensitivity of the SAR instrument, because it is pointed in the direction of the assumed slope movement. Slopes that are facing in a North or South direction may be effectively monitored with InSAR; however, the minimum detectable movement is higher for these slopes. This minimum detectable movement is determined by the slope geometry.

<u>Slope Grade:</u> Steep slopes are often difficult to monitor with InSAR due to layover, foreshortening and shadow effects. In addition, complicated topography creates a

challenge in eliminating residual topographic phase, especially when an accurate DEM is not available. Slope grades that are much less than the SAR incidence angle are preferable.

Image Coherence: The InSAR coherence is one of the main factors in determining suitability. Slopes with heavy brush, fast growing vegetation and deciduous forests are generally not suitable for InSAR monitoring unless natural or artificial (e.g., structures, corner reflectors) point targets are present.

Existing Site Data: The availability of site survey and control data, coupled with orthophotography, is very useful for maximizing the accuracy of the horizontal positioning of the InSAR data. In addition, these data help to provide a means to interpret the InSAR-derived movement information to determine the overall impact of any significant movement. The availability of a recent DEM is also important to the application of InSAR. Usable DEMs have the following specifications: 25 – 30 m spacing with vertical accuracies of 5 – 20 m. Ideally, the DEM should cover the entire region SAR image (50 × 50 km or 100 × 100 km), and minimally should cover about 5% (~125 km^2) of the SAR image.

Data Availability: New data can always be captured on sites of interest; however, the availability of a large quantity of SAR data in the historical archive will also facilitate a review of the movement history if the data are closely spaced in time and have reasonable coherence. This is particularly relevant in a project where it is required to perform an historical analysis of the movement using data available in the SAR archive.

C.6 Variations to the Application of InSAR Monitoring

There are several variations that can be made to the application of InSAR. In addition, there are several new satellites that have recently been launched which will provide enhancements over current capabilities. These are discussed below.

C.6.1 Corner reflectors:

Phase stable reflectors can serve the dual purpose of facilitating geo-referencing to site control and improving coherence in regions that are not suitable for traditional InSAR. Reflectors made from sheet and angle aluminum are robust and not generally susceptible to wind, rain or snow damage. Tests conducted in Alberta and Newfoundland, Canada, have demonstrated their ability to weather harsh environments over many years. As shown in Figure C.10, several designs are available, including those mounted with steel pegs and on concrete base foundations. The steel peg design can be field assembled and installed in about 90 minutes. There are issues specific to those techniques that include, for example, the proper sizing, placement and positioning of reflectors, the resolution of phase ambiguities in the interferograms with spatially discontinuous phase information, and the removal of artifacts such as atmospheric effects (Ferretti, et al., 2000, 2001, Werner, et al., 2003).

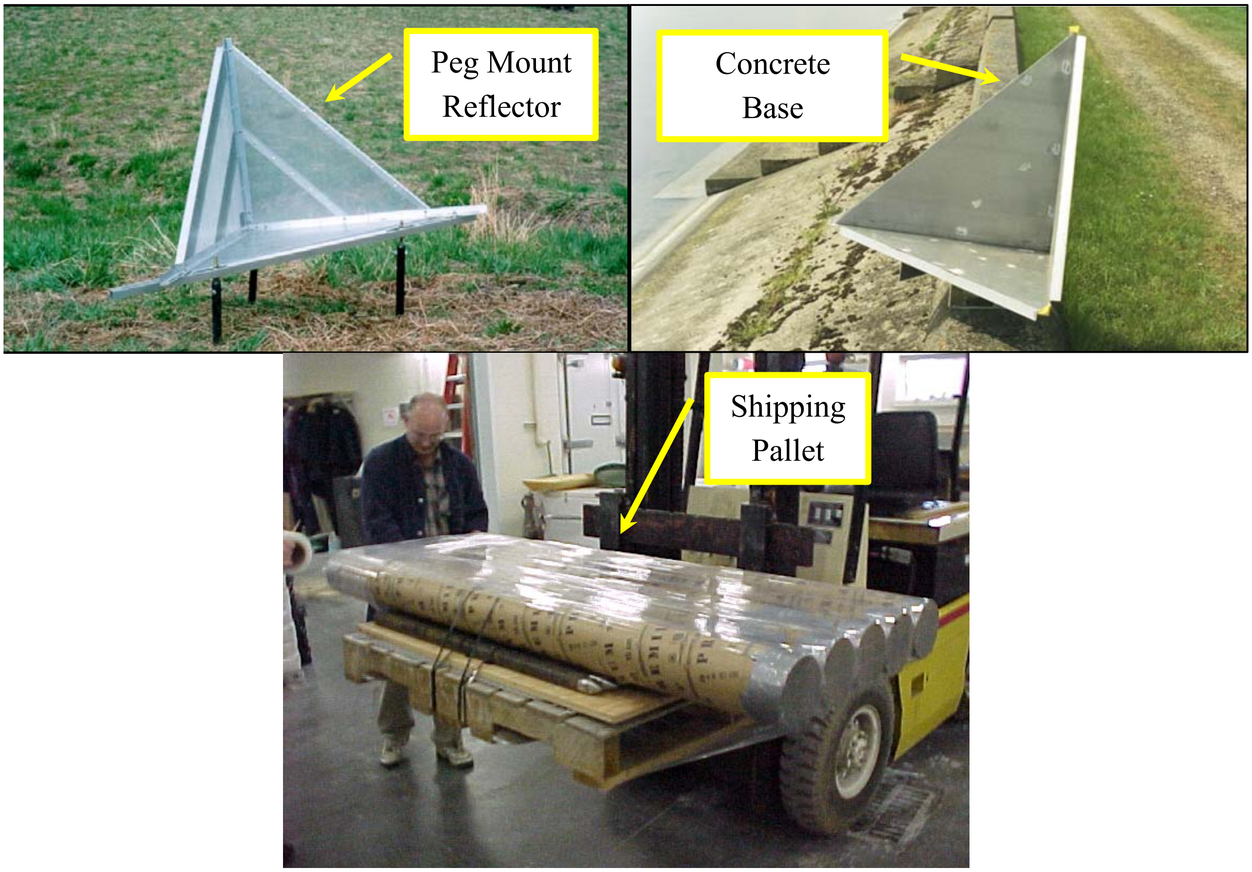


Figure C.10 Radar reflectors using two different mounts (upper left and right), and packaged for shipping (lower center)

C.6.2 Interferometric Point Target Analysis (IPTA)

IPTA and PS InSAR is finding greater use due to lower costs of European ERS and ENVISAT data and the relative success that monitoring programs have seen in producing high accuracy results (on the order of millimeters). They are typically used with historically archived data and require stacks of images of minimum 15 scenes and more typically between 25 and 35 images covering 3 to 5 year timeframes. When used in conjunction with corner reflectors, success in the application of InSAR is virtually guaranteed regardless of the site. If the ground movement behavior can be described by a mathematical model, the technique can also be used to correct for atmospheric effects and topographic errors. Figure C.11 shows an example of subsidence within an urban area as determined using the IPTA technique (GAMMA website).

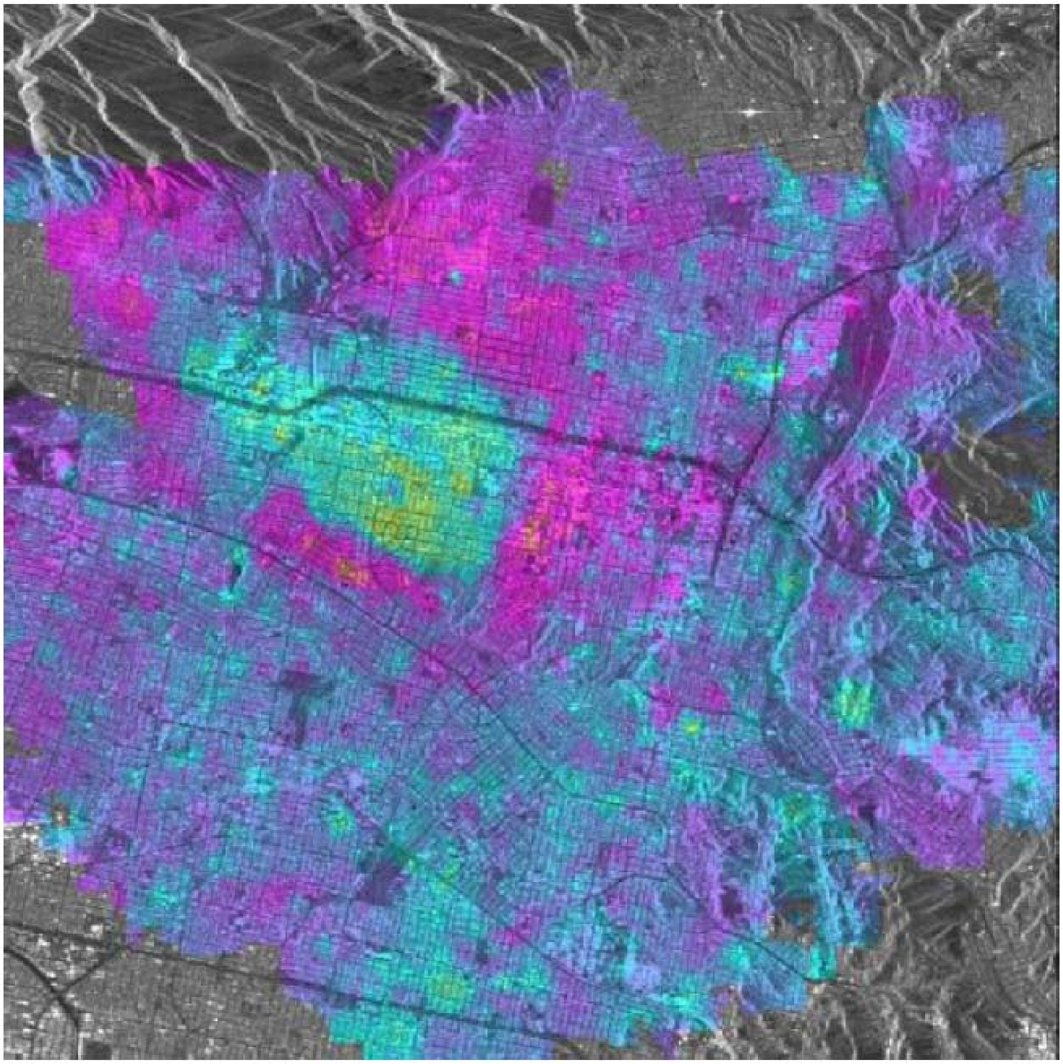


Figure C.11 IPTA example (Colour Cycle = 4 mm (0.16 inch) /year) (GAMMA website)

C.6.3 Higher Resolution Satellites

Two new high resolution SAR satellites have been launched in 2007, RADARSAT-2 and TerraSAR-X, as shown in the illustrations of C.12. RADARSAT-2 is a C-band satellite (similar to RADARSAT-1, ERS and ENVISAT) and has a maximum resolution of 3 m, with the possibility of being increased to 1 m after launch. This platform has much better orbit control than its predecessor RADARSAT-1, and consequently more of the scenes acquired for monitoring programs should be suitable for InSAR. The increased C-Band (5.4 GHz) resolution should, in theory, improve coherence due to reduced clutter levels in higher resolution cells and consequently regions that are presently not suitable for InSAR may be suitable with RADARSAT-2. TerraSAR-X has have a maximum resolution of 1 m, although it operates at X-Band (9.65 GHz) and may be less suitable for InSAR in vegetated regions compared with RADARSAT-2. The relatively high resolutions from these two satellites imply that ground motion monitoring will increase significantly due to the ability to image smaller features on the ground and thus measure greater movement details. This will be particularly relavant for monitoring smaller slopes or slopes with smaller or more complex moving features.

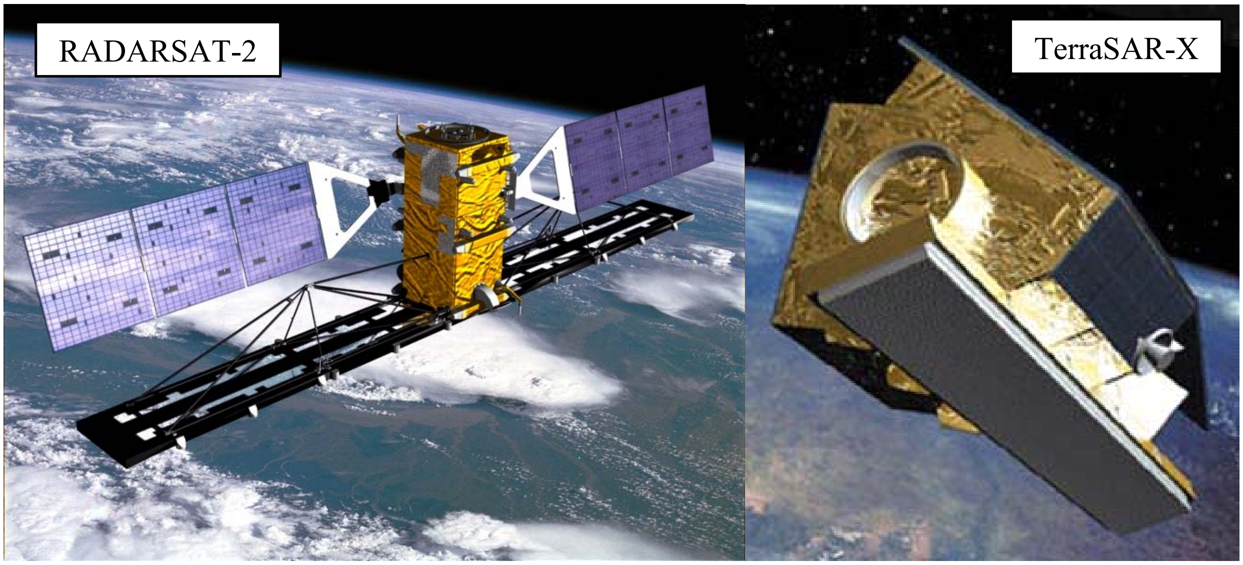


Figure C.12 The new SAR satellites RADARSAT-2 and TerraSAR-X

L-Band SAR: Advanced Land Observing Satellite (ALOS) carries an L-Band (1.27 GHz) sensor called PALSAR. It is the successor of the Japanese satellite JERS and with imaging resolutions between 7 – 44 m in Fine Mode, it is similar in resolution to RADARSAT-1 and ERS/ENVISAT. L-Band is known to be less susceptible to problems of temporal decorralation due to vegetation. Compared with C-Band (approximately 56 mm), the longer L-Band wavelength (approximately 246 mm) does not interact as much with tree canopies because the wavelength is much larger than a typical tree leaf, needle or branch structure. Consequently, certain vegetation types are transparent to the L-Band sensor, thus the SAR receives more echoes from the ground compared to the vegetation. Although there is improved overall coherence, L-Band is more susceptible to ionsphere effects than C-Band.

D. REFERENCES

1. Adam, D.P., 1975, A late Holocene pollen record from Pearson's Pond, Weeks Creek landslide, San Francisco Peninsula, California: U.S. Geological Survey Journal of Research, vol. 3, no. 6, p. 721-731.

2. Aleotti, P, and Chowdury, R., 1999, Landslide hazard assessment: summary review and new perspectives: Bulletin of Engineering Geology and the Environment, p. 21-44.

3. Alexandrowicz, S.W., and Alexandrowicz, Z., 1999, Recurrent odeling landslides—A case study of the Krynica landslide in the Polish Carpathians: The Holocene, v. 9, p. 91-99.

4. Amelung, F., Galloway, D.L., Bell, J.W., Zebker, H.A., and Laczniak, R.L., 1999, Sensing the ups and downs of Las Vegas—InSAR reveals structural control of land subsidence and aquifer-system deformation: Geology, v. 27, no. 6, p. 483–486.

5. American Society of Civil Engineers (ASCE), 1984, "Guidelines for the seismic design of oil and gas pipeline systems," Technical Council on Lifeline Earthquake Engineering, Committee on Gas and Liquid Fuel Lifelines, New York.

6. Amitrano, D., Gaffet, S., Malet, J.P., Maquaire, O., 2007, Understanding mudslides through micro-seismic monitoring: the Super-Sauze (South-East French Alps) case study, Bulletin de la Societe Geologique de France 2007 178: 149-157.

7. Angeli, M.-G., Pasuto, A., and Silvano, S., 2000, A critical review of landslide monitoring experiences, Engineering Geology, Volume 55, No. 3, p. 133-147.

8. Australian Geomechanics Society, 2000, Landslide risk management concepts and guidelines, Australian Geomechanics, 37, 54-55.

9. Baek, Y., Koo, H.B., and Bae, G.J., 2004, Study on development monitoring system of slope using the optical fiber sensor, in Lacerda, W.A., Erlich, M., Fontoura, S.A.B., and Sayao, A.S.F., eds., Landslides—Evaluation and stabilization, Proceedings of the 9th International Symposium on Landslides: London, A.A. Balkema Publishers, v. 1, p. 755-758

10. Baker, R., 2005, Variational slope stability analysis of materials with nonlinear failure criterion, The Electronic Journal of Geotechnical Engineering, v. 10, bundle A, on-line at http://www.ejge.com/2005/Ppr0514/Ppr0514.htm (accessed 3/2/2007).

11. Baron, I., Agliardi, F., Ambrosi, C., and Crosta, G.B., 2005, Numerical analysis of deep-seated mass movements in the Magura Nappe; Flysch Belt of the western Carpathians (Czech Republic).

12. Baron, I., Krejci, O., Cilek, V., Hubatka, F., 2004, Holocene history and paleo-geomorphic reconstruction of deep-seated landslides in the RAČA (flysch belt of the western Carpathians, Czech Republic): Geophysical Research Abstracts, volume 6, 06156.

13. Bauer, R.A., and Hunt, S.R., 1982, in Peng, S.S., and Harthill, M., eds., Proceedings, Workshop on Surface Subsidence due to Underground Mining: West

Virginia University, Morgantown, WV, 1981.

14. Baum, R.L., and Reid, M.E., 1995, Geology, hydrology, and mechanics of a slow-moving, clay-rich landslide, Honolulu, Hawaii, GSA Reviews in Engineering Geology X, p. 79-105.

15. Baum, R.L. and Reid, M.E., 2000, Groundwater isolation by low-permeability clays in landslide shear zones, in Bromhead, E., Dixon, N., and Ibsen, M., eds., Landslides in research, theory, and practice, Proceedings of the 8th International Symposium on Landslides, London, Thomas Telford, p. 139-144.

16. Baum, R.L., 2000, Computer programs for limit-equilibrium slope-stability analysis—FelleniusGS, BishopGS, and JanbuGS: U.S. Geological Survey Open-File Report 00-107.

17. Baum, R.L., 2003, Earth flows in Middleton, G.V., Church, M.J., Coniglio, M., Hardie, L.A., and Longstaffe, F.J., eds., Encyclopedia of Sediments and Sedimentary Rocks: Dordrect, U.K., Kluwer Academic Publishers, p. 247-248.

18. Baum, R.L., Galloway, D.L., and Harp, E.L., 2008, Landslide and sand subsidence hazards to pipelines, U.S. Geological Survey, Open-File Report 2008-1164.

19. Baum, R.L., Harp, E.L., and Hultman, W.A., 2000, Map showing recent and historic landslide activity on coastal bluffs of Puget Sound between Shilshole Bay and Everett, Washington, U.S. Geological Survey Miscellaneous Field Studies Map, MF 2346, 1 sheet, 1:24,000.

20. Baum, RL., and Johnson, A.M., 1996, Overview of landslide problems, research, and mitigation, Cincinnatti, Ohio, area, U.S. Geological Survey Bulletin 2059-A, 33 p.

21. Baum, R.L., Johnson, A.M., and Fleming, R.W., 1988, "Measurement of slope deformation using quadrilaterals," Chapter B, Bulletin 1842, U.S. Geological Survey, Denver, CO.

22. Baum, R.L., McKenna, J.P., Godt, J.W., Harp, E.L., and McMullen, S.R., 2005, Hydrologic monitoring of landslide-prone coastal bluffs near Edmonds and Everett, Washington, 2001–2004, U.S. Geological Survey Open-File Report 2005-1063, 42 p.

23. Baum, R.L., Messerich, J., and Fleming, R.W., 1998, Surface deformation of slow-moving, clay-rich landslides, Honolulu, Hawaii, Environmental & Engineering Geoscience, v.4, no. 3, p. 283-306.

24. Bawden, G., 2007, personal communication.

25. Bell, J.W., Amelung, Falk, Ramelli, A.R., and Blewitt, Geoff, 2002, Land subsidence in Las Vegas, Nevada, 1935-2000, New geodetic data show evolution, revised spatial patterns, and reduced rates, Environmental and Engineering Geoscience, v. III, no. 3, p. 155–174.

26. Beneveli, R.M., Carvalho, M.O.M., and Carvalho, J.C., 2004, Using a thermal sensor to determine the soil matric suction of a gully slope, in Lacerda, W.A., Erlich, M., Fontoura, S.A.B., and Sayao, A.S.F., eds., Landslides—Evaluation and

stabilization, Proceedings of the 9th International Symposium on Landslides, London, A.A. Balkema Publishers, v. 1, p. 767-772.

27. Bianchi, J.C., and Farrington, S.P., 2001, Direct push monitoring point assessment, Air Force Research Laboratory Report AFRL-ML-TY-TP-2000-4535, 715 p.

28. Bishop, A.W., 1959, The principle of effective stress, Teknisk Ukeblad, v. 39, p. 859-863.

29. Blake, T.F., Hollingsworth, R.A., and Stewart, J.P., 2002, Recommended procedures for implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California, Southern California Earthquake Center, 120 p.

30. Bogaard, T.A., Antoine, P., Desvarreux, P., Giraud, A., and van Asch, Th. W. J., 2000, The slope movements within the Mondores graben (Drome, France); the interaction between geology, hydrology and typology, Engineering Geology, vol. 55, No. 4, p. 297-312.

31. Borchers, J. W., 1994, Characterization of fractures in granitic rocks at Wawona, Yosemite National Park, California; a comparison of borehole geophysical and downhole visualization tools, in Lawson, C.A. and Bennett, P.C., eds., Scientific Visualization Workshop, U.S. Geological Survey Open-File Report 94-134.

32. Borchers, J.W., Gerber, M., Wiley, J., and Mitten, H.T., 1998, Using down-well television surveys to evaluate land subsidence damage to water wells in the Sacramento Valley, California, in Borchers, J.W., ed., Land subsidence — Case studies and current research, Proceedings of the Dr. Joseph F. Poland Symposium on Land Subsidence, Association of Engineering Geologists Special Publication 8, p. 89–105.

33. Brien D.L., Reid M.E., 2001, 3-D slope stability of coastal bluffs incorporating 3-D pore pressures, Seattle, WA, Transactions of the American Geophysical Union, 82(47), F410

34. Brien, D.L., and Reid, M.E., 2007, Modeling 3-D slope stability of coastal bluffs using 3-D ground-water flow, Southwestern Seattle, Washington, U.S. Geological Survey Scientific Investigations Report 2007-5092, 54 p.

35. Bromhead, E.N., 2004, Landslide slip surfaces—Their origins, behaviour and geometry, in Lacerda, W.A., Erlich, M., Fontoura, S.A.B., and Sayao, A.S.F., eds., Landslides—Evaluation and stabilization, Proceedings of the 9th International Symposium on Landslides, London, A.A. Balkema Publishers, v. 1, p. 3-22.

36. Bromhead, E.N., Ibsen, M-L., Papanastassiou, X., and Zemichael, A.A., 2002, Three-dimensional stability analysis of a coastal landslide at Hanover Point, Isle of Wight, Quarterly journal of Engineering Geology and Hydrogeology, v. 35, p. 79-88.

37. Brückl, E., Brunner, F.K. and Kraus, K., 2006, Kinematics of a deep-seated landslide derived from photogrammetric, GPS and geophysical data, Engineering Geology, Volume 88, No. 3-4, 15 December 2006, p. 149-159.

38. Bull, W.B., King, J., Kong, F., Moutoux, T., Phillips, W.M., 1994, Lichen dating of co-seismic landslide hazards in alpine mountains, Geomorphology, v. 10, p. 253-264.

39. Butler, R.C., 2007, written communication.

40. Cala, M., Flisiak, J, and Tajdus, A., 2004, Slope stability analysis with modified shear strength reduction technique, in Lacerda, W.A., Erlich, M., Fontoura, S.A.B., and Sayao, A.S.F., eds., Landslides—Evaluation and stabilization, Proceedings of the 9th International Symposium on Landslides, London, A.A. Balkema Publishers, v. 2, p. 1085-1089.

41. Campbell Scientific, 2007, Time-domain reflectometry, http://www.campbellsci.com/time-domain-reflectometry (accessed March, 2007)

42. Canadian Geotechnical Society, 2006, Canadian Foundation Engineering Manual (4th ed.), Richmond, B.C., Canada, BiTech Publisher Ltd., 504 p.

43. Carpenter, M.C., 1999, South-central Arizona—Earth fissures and subsidence complicate development of desert water resources, in Galloway, D.L., Jones, D.R., and Ingebritsen, S.E., eds., 1999, Land subsidence in the United States, U.S. Geological Survey Circular 1182, p. 65-78, http://pubs.usgs.gov/circ/circ1182/pdf/09Arizona.pdf, accessed May 23, 2007.

44. Carrera, A., 1983, Multivariate models for landslide hazard evaluation, Mathematical Geology, v. 15, no. 3, p. 403-426.

45. Carrera, A., 1989, Landslide hazard mapping by statistical methods: a “black-box” model approach, in Siccardi, F., and Bras, R.L., eds., Proceedings Natural Disasters in European-Mediterranean Countries, Perugia 27th June-1 July, CNR-U.S.N.S.F., p. 205-224.

46. Carrera, A., Cardinali, M., Detti, R., Guzzetti, F., Pasqui, V., and Reichenbach, P., 1991, GIS techniques and statistical models in evaluating landslide hazard, Earth Surface Processes and Landforms, v. 16, p. 427-445.

47. Casson, B., Delacourt, C., and Allemand, P., 2005, Contribution of multi-temporal remote sensing images to characterize landslide slip surface — Application to the La Clapière landslide (France), Natural Hazards and Earth System Sciences, vol. 5, p. 425-437.

48. C-CORE, 1998, Large scale □odeling of soil/pipe interaction under moment loading — Laboratory testing report, contract report for the Geological Survey of Canada, C-CORE Publication 98-C23.

49. C-CORE, 2003, Extended model for pipee soil interaction, final report prepared for Pipeline Research Council International, C-CORE Report R-02-044-113, August.

50. C-CORE, 2008, Pipeline integrity for ground movement hazards, report prepared for Pipeline Research Council International and the U.S. Department of Transportation, C-CORE Report R-07-082-459.

51. Chelli, A., Mandrone, G., Ruffini, A., and Truffelli, G., 2005, Dynamics and conceptual model of the Rossena castle landslide (Northern Apennines, Italy),

Natural Hazards and Earth System Sciences, vol. 5, 903-909.

52. Chen, R.H., and Chameau, J.-L., 1982, Three-dimensional limit equilibrium analysis of slopes, Geotechnique, v. 32, no 1., p. 31-40.

53. Chleborad, A.F., 1996, Radiocarbon age of a newly identified Slumgullion landslide deposit, in Varnes, D.J., and Savage, W.Z., editors, The Slumgullion earth flow—A large-scale natural laboratory, U.S. Geological Survey Bulletin 2130, ch. 5, p. 29-33.

54. Chugh, A.K., Stark, T.D., and DeJong, K.A., 2007, Reanalysis of a municipal landfill slope failure near Cincinnati, Ohio, USA, Canadian Geotechnical Journal, v. 44, p. 33-53, DOI: 10.1139/T06-089.

55. Coe, J.A., Ellis, W.L., Godt, J.W., Savage, W.Z., Savage, J.E., Michael, J.A., Kibler, J.D., Powers, P.S., Lidke D.J., and Debray, S., 2003, Seasonal movement of the Slumgullion landslide determined from Global Positioning System surveys and field instrumentation, July 1998-March 2002, Engineering Geology, Volume 68, No. 1-2, p. 67-101.

56. Coe, J.A., Godt, J.W., Baum, R.L., Buckram, R.C., and Michael, J.A., 2004, Landslide susceptibility from topography in Guatemala, in Lacerda, W.A., Erlich, M., Fonterra, S.A.B., and Sayao, A.S.F., eds., Landslides—Evaluation and stabilization, Proceedings of the 9th International Symposium on Landslides, London, A.A. Balkema Publishers, v. 1, p. 69-78.

57. Coe, J.A., Godt, J.W., Parise, M, and Moscariello, A., 2003, Estimating debris-flow probability using debris-fan stratigraphy, historic records, and drainage-basin morphology, Interstate 70 Highway Corridor, Central Colorado, in Rickenmann, D.and Chen, C., eds., Debris flow hazards mitigation: mechanics, prediction, and assessment, Proceedings of the Third International Conference on Debris-Flow Hazards Mitigation, Millpress, Rotterdam, p. 1085-1096.

58. Coe, J.A., Harp, E.L., Tarr, A.C., Michael, J.A., 2005, Rock-fall hazard assessment of Little Mill campground, American Fork Canyon, Uinta National Forest, Utah, U.S. Geological Survey Open File Report 2005-1229, 48 p., two 1:3000-scale plates.

59. Coe, J.A., Harp, E.L., Tarr, A.T., and Michael, J.A., 2007, Rockfall hazard at Little Mill Campground, Uinta National Forest, Part 1: Geologic hazard assessment, in Schaefer, V.R., Schuster, R.L., and Turner, A.K., (eds.), Conference Presentations, 1st North American Landslide Conference, Vail, Colorado, AEG Special Publication 23, Association of Environmental & Engineering Geologists, CD-ROM, 14 p.

60. Colesanti, Carlo, and Wasowski, Janusz, 2006, Investigating landslides with space-borne Synthetic Aperture Radar (SAR) interferometry, Engineering Geology, Volume 88, No. 3-4, 15 December, p. 173-199.

61. Collins, B.D., Kayen, Robert, Reiss, Thomas, and Sitar, Nicholas, 2007, Terrestrial LIDAR Investigation of the December 2003 and January 2007 Activations of the Northridge Bluff Landslide, Daly City, California, U.S. Geological Survey Open-

File Report 2007-1079.

62. Corominas, Jordi, Moya, José, Ledesma, Alberto, Lloret, Antonio, Gili, J.A., 2005, prediction of ground displacements and velocities from groundwater level changes at the Vallcebre landslide (Eastern Pyrenees, Spain), Landslides, vol. 2, p. 83-96.

63. Corsini, A., Borgatti, L., Caputo G., De Simone, N., Sartini, G., and Truffelli, G., 2006, Investigation and monitoring in support of the structural mitigation of large slow moving landslides: an example from Ca' Lita (Northern Apennines, Reggio Emilia, Italy), Natural Hazards and Earth System Sciences, volume 6, p. 55-61.

64. Crosta, G.B., Imposimato, S., and Roddeman, D.G., 2003, Numerical modeling of large landslides stability and runout, Natural Hazards and Earth System Sciences v. 3 p. 523–538.

65. Cruden, D.M. and Varnes, D.J., 1996, Landslide types and processes, Chapter 3 in Landslides—Investigations and mitigation, Turner, A.K., and Schuster, R.L. eds., Transportation Research Board, Special Report 247, p. 36-75.

66. Curlander, J.C., and McDonough, R.N., 1991, Synthetic Aperture Radar: Systems and Signal Processing, New York, J. Wiley & Sons.

67. Davies, W.E., and LeGrand, H.E., 1972, Karst of the United States, in Herak, M. and Stringfield, V.T., eds., Karst—Important karst regions of the northern hemisphere, New York, Elsevier Publishing Co., p. 467–505.

68. DeGraff, J.V., Cannon, S.H., Gartner, J.E., and Gallegos, A.J., 2007, Reducing post-wildfire debris flow risk through the burned area emergency response (BAER) process, in Schaefer, V.R., Schuster, R.L., and Turner, A.K., (eds.), Conference Presentations, 1st North American Landslide Conference, Vail, Colorado, AEG Special Publication 23, Association of Environmental & Engineering Geologists, CD-ROM, 10 p.

69. Denlinger, R.P., and Iverson R.M., 2004, Granular avalanches across irregular three-dimensional terrain—Theory and computation, Journal of Geophysical Research, vol. 109, F01014.

70. Dixon, N., and Bromhead, E.N., 2002, Landsliding in London clay coastal cliffs, Quarterly Journal of Engineering Geology and Hydrogeology, vol. 35 p. 327-343.

71. Dowding, C.H., Su, M.B., and O'Connor, K., 1989, Measurement of rock mass deformation with grouted coaxial cables, Rock Mechanics and Engineering, v. 22, p. 1-22.

72. Duncan, J.M., 1996, Soil slope stability analysis, Chapter 13, in Landslides—Investigation and mitigation, Turner, A.K., and Schuster, R.L. eds., Transportation Research Board, Special Report 247, p. 337-371.

73. Duncan, J.M., 2000, Factors of safety and reliability in geotechnical engineering, Journal of Geotechnical and Geoenvironmental Engineering, American Society of Civil Engineers, p. 307-316.

74. Dunrud, C.R., 1984, Coal mine subsidence—western United States, in Holzer, T.L., ed., Man-induced land subsidence, Geological Society of America Reviews in

Engineering Geology, v. 6, p. 151–194.

75. Duranthon, J.P., 2004, The landslide of “Ruines de Séchilienne” ground radar for monitoring of landslides, in Lacerda, W.A., Erlich, M., Fontoura, S.A.B., and Sayao, A.S.F., eds., Landslides—Evaluation and stabilization, Proceedings of the 9th International Symposium on Landslides, London, A.A. Balkema Publishers, v. 1, p. 635-641.

76. Ellis, W.L., Priest, G.R., and Schulz, W.H., 2007, Precipitation, pore pressure, and landslide movement—Detailed observations at the Johnson Creek landslide, coastal Oregon, in Schafer, V.R., Schuster, R.L., and Turner, A.K., Conference Presentations, 1st North America Landslide Conference, Vail, Colorado, Association of Environmental and Engineering Geologists Special Publication 23, (CD-ROM).

77. Ellis, W.L., 2007, oral communication.

78. El-Ramly, H., Morgenstern, N.R., and Cruden, D.M., 2002, Probabilistic slope stability analysis for practice, Canadian Geotechnical Journal, p. 665-683.

79. Farina, P., Colombo, D., Fumagalli, A., Marks, F., and Moretti, S., 2006, Permanent scatterers for landslide investigations: outcomes from the ESA-SLAM project, Engineering Geology, Volume 88, No. 3-4, p. 200-217.

80. Fenton, G.A., and Griffiths, D.V., 1993, Statistics of block conductivity through a simple bounded stochastic medium, *Water Resources Research*, **29**(6), 1825--1830.

81. Ferretti, A., Prati, C. and Rocca, F., 2000, Nonlinear subsidence rate estimation using permanent scatters in differential SAR interferometry, IEEE Transactions on Geoscience and Remote Sensing, vol. 38, no.5, p. 2202-2212, Sept..

82. Ferretti, A., Prati, C. and Rocca, F., 2001, Permanent scatters in SAR interferometry, IEEE Transactions on Geoscience and Remote Sensing, vol. 39, no.1, p. 8-20, January.

83. Fleming, R.W., Johnson, R.B., and Schuster, R.L., 1988, The reactivation of the Manti landslide, Utah, Chapter A in The Manti, Utah, landslide, U.S. Geological Survey Professional Paper 1311, p. 1-22.

84. Fraser, C.S., and Gruendig, L. 1985, The analysis of photogrammetric deformation measurements on Turtle Mountain, Photogrammetric Engineering and Remote Sensing v. 51, no. 2, p. 207-216.

85. Fredlund, D.G., and Rahardjo, H., 1993, Soil mechanics for unsaturated soils, New York, Wiley, 517 p.

86. Freeze, R.A., and Cherry, J.A., 1979, Groundwater, Prentice-Hall, Inc., Englewood Cliffs, N.J.

87. Froese, C.R., Keegan, T.R., Cavers, D.S., and van der Kooij, M., 2005, Detection and monitoring of complex landslides along the Ashcroft rail corridor using spaceborne InSAR, Hungr, O., Fell, R., Couture, R., and Bernhard, E., eds., Landslide Risk Management, Proceedings of the 2005 International Conference on

Landslide Risk Management, New York, A.A. Balkema, p. 565-570.

88. Fruneau, B. and Sarti, F., 2000, Detection of ground subsidence in the city of Paris using radar interferometry: isolation of deformation from atmospheric artifacts using correlation, Geophysical Research Letters, vol. 27, no. 24, p.3981-3984.

89. Fukuzono, T., 1990, Recent studies on time prediction of slope failure, Landslide News, v. 4, p. 9–12.

90. Gaertner, G., Pohl,W., Lindner, H., 2000, Monitoring and modeling of post-failure behavior of a large overburden slide in the lignite mine of Zwenkau (Saxony, Germany), in Bromhead, E., Dixon, N., and Ibsen, M, editors, Landslides in Research, Theory and Practice, volume 2, Proceedings of the Eighth International Symposium on Landslides, London, Thomas Telford, p. 603-608.

91. Galloway, D.L., Jones, D.R., and Ingebritsen, S.E., eds., 1999, Land subsidence in the United States, U.S. Geological Survey Circular 1182, 177 p., http://pubs.usgs.gov/circ/circ1182/, accessed March 28, 2007.

92. Gamma Remote Sensing, http://www.gamma-rs.ch/

93. Gatelli, F., Monti Guarnieri, A., Parizzi, F., Pasquali, P., Prati, C. and Rocca, F., 1994, The wavelength shift in SAR interferometry, IEEE Transactions on Geoscience and Remote Sensing, vol. 32, no.4, p. 855-865, July 1994.

94. Gili, J.A. Corominas, Jordi, and Rius, Joan, 2000, Using Global Positioning System techniques in landslide monitoring, Engineering Geology, Volume 55, No. 3, February 2000, p. 167-192.

95. Gomberg, J.S., Bodin, P.W., Savage, W.Z., Jackson, M.E., 1996, Slidequakes and fault creep at the Slumgullion landslide—An analog to crustal tectonics, in Varnes, D.J., and Savage, W.Z., editors, The Slumgullion earth flow—A large-scale natural laboratory, U.S. Geological Survey Bulletin 2130, ch. 14, p. 85 - 91.

96. Gray, R.E., and Bruhn, R.W., 1984, Coal mine subsidence—Eastern United States, in Holzer, T.L., ed., Man-induced land subsidence, Geological Society of America Reviews in Engineering Geology, v. 6, p. 123–149.

97. Greif, V., Sassa, K., and Fukuoka, H., 2004, Monitoring of rock displacements at Bitchu-Matsuyama rock slope in Japan using linear variable differential transformer (LVDT) sensors, in Lacerda, W.A., Erlich, M., Fontoura, S.A.B., and Sayao, A.S.F., eds., Landslides—Evaluation and stabilization, Proceedings of the 9th International Symposium on Landslides, London, A.A. Balkema Publishers, v. 1, p. 773-779.

98. Griffiths, D.V., and Fenton, G.A., 1993, Seepage beneath water retaining structures founded on spatially random soil, Geotechnique, 43(6), 577-587.

99. Griffiths, D.V., and Fenton, G.A., 2004, Probabilistic slope stability by finite elements, ASCE Journal of Geotechnical and Geoenvironmental Engineering, 130(5), 507—518.

100. Griffiths, D.V., and Lane, P.A., 1999, Slope stability analysis by finite elements,

Geotechnique vol. 49, no.3, p. 387-403.

101. Griffiths, D.V., and Marquez, R.M., in press, Three-dimensional slope stability analysis by elasto-plastic finite elements, Accepted for publication by Geotechnique

102. Grivas, D.A., Chakravarthy, B. Schultz, B.A., McGuffey, V.C., O'Neil, G.D., and Simmonds, G.R., 1996, Achieving reliable designs for pipelines traversing unstable slopes, in Proceedings of the 15th International Conference on Offshore Mechanics and Arctic Engineering, Florence, Italy, June

103. Grivas, D.A., Schultz, B.A., O'Neil, G.D., and Simmonds, G.R., 1996, Phenomenological models to predict rainfall-induced ground movements, in Proceedings of the 15th International Conference on Offshore Mechanics and Arctic Engineering, Florence, Italy, June.

104. Guzzetti, F., Crosta, G., Detti, R., and Agliardi, F., 2002, STONE: a computer program for the three-dimensional simulation of rock-falls, Computers and Geosciences, 28, 9, 1079–1093.

105. Hammond, C., Hall, D., Miller, S., and Swetik, P., 1992, Level I stability analysis (LISA) documentation for version 2, General Technical Report INT-285, USDA Forest Service Intermountain Research Station, 121 p.

106. Haneberg, W.C., 2004, A rational probabilistic method for spatially distributed landslide hazard assessment, Environmental and Engineering Geoscience, vol. X, no. 1, p. 27-43.

107. Harp, E.L, Michael, J.A., and Laprade, W.T., 2006, Shallow landslide hazard map of Seattle, Washington, U.S. Geological Survey Open-File Report 2006-1139, 20 p., 2 pl.

108. Harp, E.L. and Noble, M.A, 1993, An engineering rock classification to evaluate seismic rock-fall susceptibility and its application to the Wasatch Front, Bulletin of the Association of Engineering Geologists, v. 30, p. 293-319.

109. Hart, J.D., Zulfiqar, N., and Popelar, C.H., 2008, Use of pipeline geometry monitoring to assess pipeline condition, report prepared for Pipeline Research Council International and the U.S. Department of Transportation.

110. Haugerud, R.A., Harding, D.J., Johnson, S.Y., Harless, J.L., Weaver, C.S., and Sherrod, B.L., 2003, High-resolution LIDAR topography of the Puget lowland, Washington, GSA Today 13, (6) (Jun), 4-10, http://www.gsajournals.org (accessed August 7, 2007).

111. Highland, L.M., 2004, Landslide types and processes, US Geological Survey Fact Sheet 2004-3072, 4 p.

112. Hillel, D., 1982, Introduction to soil physics, San Diego, Academic Press, 364 p. 1980

113. Hinzman, L.D., Bettez, N.D., Bolton, W.R., Chapin, F.S., Dyurgerov, M.B., Fastie, C.L., Griffith, B., Hollister, R.D., and others, 2005, Evidence and implications of recent climate change in northern Alaska and other Arctic regions, Climate Change,

v. 72 (3), doi: 10.1077/s10584-005-5352-2, p. 251–298.

114. Hiura, H., Furuya, G, Fukuoka, H., and Sassa, K., 2000, Investigation of the groundwater distribution in crystalline schist landslide Zentoku, Shikoku Island, Japan,), in Bromhead, E., Dixon, N., and Ibsen, M., editors, Landslides in Research, Theory and Practice, volume 2, Proceedings of the Eighth International Symposium on Landslides, London, Thomas Telford, p. 719-724.

115. Holzer, T.L., 1984, Ground failure induced by ground-water withdrawal from unconsolidated sediment, in Holzer, T.L., ed., Man-induced land subsidence, Geological Society of America Reviews in Engineering Geology, v. 6, p. 67–105.

116. Holzer, T.L., and Gabrysch, R.K., 1987, Effect of water-level recoveries on fault creep, Houston, Texas, Ground Water, v. 25, no. 4, p. 392–397.

117. Holzer, T.L., and Galloway, D.L., 2005, Impacts of land subsidence caused by withdrawal of underground fluids in the United States, in Ehlen, J., Haneberg, W.C., and Larson, R.A., eds., Humans as geologic agents, Geological Society of America Reviews in Engineering Geology, v. 16, p. 87–99, doi: 10.1130/2005.4016(08).

118. Honegger, D.G. and Nyman, D.J., 2004, Seismic Design and Assessment of Natural Gas and Liquid Hydrocarbon Pipelines, Pipeline Research Council International, Inc., No. L51927.

119. Honegger D.G., Wijewickreme, D., and H. Karimian, 2006, Assessment of geosynthetic fabrics to reduce soil loads on buried pipelines, report to PRCI Inc. under contract PR-268-03111.

120. Honegger, D.G., 1999, Field measurement of axial soil friction on buried pipelines," Proceedings of the 5th National Conference on Lifeline Earthquake Engineering, American Society of Civil Engineers.

121. HRB Singer, Inc., 1977, Nature and distribution of subsidence problems affecting HUD and urban areas, U.S. Department of Housing and Urban Development, 113 p. (Available from NTIS, PB 80172778).

122. Hungr, O., and Morgenstern, N.R., 1984, High velocity ring shear tests on sand, Geotechnique, V. 34, p. 415-421.

123. Hungr, O., Corominas, J., and Eerhardt, E., 2005, Estimating landslide motion mechanism, travel distance, and velocity, in Hungr, O., Fell, R., Couture, R., and Bernhard, E., eds., Landslide risk management, Proceedings of the 2005 International Conference on Landslide Risk Management, New York, A.A. Balkema, p. 99-128.

124. Hungr, O., Salgado, F.M., and Byrne, P.M., 1989, Evaluation of a three-dimensional method of slope-stability analysis, Canadian Geotechnical Journal, v. 26, p. 679-686.

125. Hürlimann, M., Ledesma, A., Corominas, J., and Prat, P., 2004, Numerical modeling of a large, deep-seated slope deformation at Encampadana, Andorra, in , in Lacerda, W.A., Erlich, M., Fontoura, S.A.B., and Sayao, A.S.F., eds.,

Landslides—Evaluation and stabilization, Proceedings of the 9th International Symposium on Landslides, London, A.A. Balkema Publishers, v. 2, p. 1073-1078.

126. Husaini, Omar, and Ratnasamy, M., 2001, An early warning system for active landslides, Quarterly Journal of Engineering Geology and Hydrogeology, v. 34, p. 299-305.

127. Hutchinson, J.N., 1983, Methods of locating slip surfaces in landslides, Bulletin of the Association of Engineering Geologists, v. 20, no. 3, p. 235-252.

128. Ingebritsen, S.E., Sanford, W.E., and Neuzil, C.E., 2006, Groundwater in geologic processes, 2nd edition, New York, Cambridge Univ. Press, 536 p.

129. Iverson R.M., 1986, Unsteady, nonuniform landslide motion—Linearized theory and the kinematics of transient response, Journal of Geology, vol. 94, p. 349 - 364.

130. Iverson R.M., 2000, Landslide triggering by rain infiltration, Water Resources Research, vol. 36, no. 7, p. 1897- 1910.

131. Iverson, R.M., Logan, M., and Denlinger, R.P., 2004, Granular avalanches across irregular three-dimensional terrain, 2. Experimental tests, Journal of Geology, v. 58, p. 227-289.

132. Jibson, R.W., Harp, E.L., and Michael, J.A., 2000, A method for producing digital probabilistic seismic landslide hazard maps, Engineering Geology, v. 58, p. 271-289.

133. Johnson, P.L., and Cole, W.F., 2001, The use of large-diameter boreholes and downhole logging methods in landslide investigations, Ferriz, Horacio and Anderson, Robert, eds., Engineering Geology Practice in Northern California, p. 95-106.

134. Johnson, W., and Miller, G.C., 1979, Abandoned coal-mined lands; nature, extent, and cost of reclamation, U.S. Department of the Interior, Bureau of Mines, 20 p.

135. Jones, C.L., Higgins, J.D., and Andrew, R.D., 2000, Colorado rockfall simulation program (CRSP), Version 4.0, March 2000, Colorado Geological Survey Report MI 66.

136. Jones, L.D., 2006, Monitoring landslides in hazardous terrain using terrestrial LiDAR: an example from Montserrat, Quarterly Journal of Engineering Geology and Hydrogeology 39: p. 371-373

137. Jotisankasa, A, Coop, M, Ridley, A, 2007, The development of a suction control system for a triaxial apparatus, Geotechnical Testing Journal,Volume 30, Issue 1, Paper ID: GTJ100026, DOI: 10.1520/GTJ100026

138. Kääb, A., 2002, Monitoring high-mountain terrain deformation from repeated air- and spaceborne optical data: examples using digital aerial imagery and ASTER data, ISPRS Journal of Photogrammetry and Remote Sensing 57 (2002), p. 39–52.

139. Kalaugher, P.G., Hodgson, R.L.P., Grainger, P., 2000, Pre-failure strains as precursors of sliding in a coastal mudslide, Quarterly Journal of Engineering Geology and Hydrogeology, v. 33, p. 325 -- 334.

140. Kane, W.F., 2007, Railway alert system: The Campbell update, Campbell Scientific Inc., Logan, Utah, v. 18, no. 1, p. 5-6.

141. Kane, W.F., and Beck, T.J., 1994, Development of a time domain reflectometry system to monitor landslide activity, Proceedings of the 45th Highway Geology Symposium, p. 163-173.

142. Keaton, J.R., and DeGraff, J.V., 1996, Surface observation and geologic mapping, Chapter 9, in Landslides—Investigations and mitigation, Turner, A.K., and Schuster, R.L. eds., Transportation Research Board, Special Report 247, p. 179-230.

143. Keaton, J.R., and Gailing, R.W, 2004, Monitoring slope deformation with quadrilaterals for pipeline risk management, Proceedings of the International Pipeline Conference 2004, Calgary, Alberta, October 2004, Paper No. IPC04-0197, 6 p.

144. Keefer, D.K., and Johnson, A.M., 1983, Earth flows–Morphology, mobilization and movement, U.S. Geological Survey Professional Paper 1264, 56 p.

145. Kennedy, R.P., Chow, A.W., and Williamson, R.A., 1977, Fault movement effects on buried oil pipeline, Journal of the Transportation Engineering Division, American Society of Civil Engineers, vol. 103, no. TE5, p. 617-633.

146. Kilburn, C. R. J., and Petley, D. N., 2003, Forecasting giant, catastrophic slope collapse—Lessons from Vajont, northern Italy, Geomorphology, v. 54, p. 49–62.

147. Köhler, H.J. and Schulze, R., 2000, Landslides triggered in clay soils -- Geotechnical measurements and calculations, in Bromhead, E., Dixon, N., and Ibsen, M.L., editors, Landslides in Research, Theory and Practice, volume 2, Proceedings of the Eighth International Symposium on Landslides, London, Thomas Telford, p. 837-842.

148. Konietzky, H., Lorenz, K., and Witter, W., 2004, Complex 3D landslide simulation, in Lacerda, W.A., Erlich, M., Fontoura, S.A.B., and Sayao, A.S.F., eds., Landslides—Evaluation and stabilization, Proceedings of the 9th International Symposium on Landslides, London, A.A. Balkema Publishers, v. 2, p. 1053-1059.

149. LaHusen, R.G., and Reid, M.E., 2000, A versatile GPS system for monitoring deformation of active landslides and volcanoes, Transactions of the American Geophysical Union, v.81, n.48, p.F320.

150. Lambe, T.W. and Whitman, R.V., 1969, Soil mechanics, New York, John Wiley, 553 p.

151. Lee, E.M., and Jones D.K.C., 2004, Landslide risk assessment, Thomas Telford Publishing, London.

152. Leroueil, S., 2001, Natural slopes and cuts—Movement and failure mechanisms, Geotechnique, vol. 51 number 3, p. 197-243.

153. Leva, D., Nico, G., Tarchi, D., Fortuny-Guasch, J. and Sieber, A.J., 2003, SAR temporal analysis of landslide by means of a ground-based SAR interferometer, IEEE Transactions on Geoscience and Remote Sensing, vol. 41, no. 1, pt.1, p.745-

752.

154. Lin, Z, and Liang, W., 1982, Engineering properties and zoning of loess and loess-like soils in China, Canadian Geotechnical Journal, Issue 19, p.76–91.

155. Lollino, G., Arattano, M., Allasia, P., and Giordan, D., 2006, Time response of a landslide to meteorological events, Natural Hazards Earth Systems. Sci., 6, 179-184, 2006

156. Lu, N. and Likos, W.J., 2006, Suction stress characteristic cure for unsaturated soil, Journal of Geotechnical and Geoenvironmental Engineering, American Society of Civil Engineers, v. 132, no. 2, p. 131-142.

157. Luzi, G.; Pieraccini, M.; Mecatti, D.; Noferini, L.; Guidi, G.; Moia, F.; Atzeni, C., 2004, Ground-based radar interferometry for landslides monitoring: atmospheric and instrumental decorrelation sources on experimental data, IEEE Transactions on Geoscience and Remote Sensing, Volume 42, No. 11, p. 2454 - 2466 DOI:10.1109/TGRS.2004.836792

158. Madole, R.F., 1996, Preliminary chronology of the Slumgullion landslide, Hillsdale County, Colorado, in Varnes, D.J., and Savage, W.Z., editors, The Slumgullion earth flow—A large-scale natural laboratory, U.S. Geological Survey Bulletin 2130, ch. 1, p. 5-7.

159. Markland, J.T., 1972, A useful technique for estimating the stability of rock slopes when the rigid wedge slide type of failure is expected, Imperial College Rock Mechanics Research Reprints, n. 19.

160. Martinez, J.D., Johnson, K.S., and Neal, J.T., 1998, Sinkholes in evaporite rocks, American Scientist, v. 86, p. 38–51.

161. Massonnet, D. and Feigl, K.L., 1998, Radar interferometry and its application to changes in the earth’s surface, Reviews of Geophysics, vol. 36, no. 4, p. 441-500, Nov. 1998.

162. McCalpin, J.P., 1984, Preliminary age classification of landslides for inventory mapping, Proceedings of the 1984 Symposium on Engineering Geology and Soils Engineering, Boise Idaho, p. 99-111.

163. McGuffey, Verne C., Modeer, Victor, A., Jr, and Turner, Keith A., 1996, Subsurface Exploration, Chapter 10 in Landslides—Investigation and mitigation, Turner, A.K., and Schuster, R.L. eds., Transportation Research Board, Special Report 247, p. 231-277.

164. Mikkelsen, P.E., 1996, Field Instrumentation, Chapter 11 in Landslides—Investigations and mitigation, Turner, A.K., and Schuster, R.L. eds., Transportation Research Board, Special Report 247, p. 278-316.

165. Mikkelsen, P.E., 2002, Cement-bentonite grout backfill for borehole instruments, Geotechnical Instrumentation News, December, p. 38-42.

166. Miller, D.J., and Sias, J., 1997, Deciphering large landslides: linking hydrological groundwater and slope stability models, Hydrological Processes, v. 12, p. 923-941.

167. Miller, D.J., 1995, Coupling GIS with physical models to assess deep-seated landslide hazards, Environmental and Engineering Geoscience, v.I, no. 3, p. 263-276.

168. Miller, G.A., and Hamid, T.B., 2007, Interface direct shear testing of unsaturated soil, Geotechnical Testing Journal, Volume 30, Issue 3, Paper ID: GTJ13301, DOI: 10.1520/GTJ13301

169. Mohareb, M.E., Elwi, A.E., Kulak, G.L., and Murray, D.W., 1994, Deformational behavior of line pipe, Structural Engineering Report 202, Department of Civil Engineering, University of Alberta, Canada.

170. Montgomery, D.R., and Dietrich, W.E., 1994, A physically based model for the topographic control on shallow landsliding, Water Resources Research, v. 30, no. 4, p. 1153-1171.

171. Moore, K., 2006, personal communication.

172. Nadim, F., Einstein, H., Roberds, W., 2005, Probabilistic stability analysis for individual slopes in soil and rock, in Hungr, O., Fell, R., Couture, R., and Bernhard, E., eds., Landslide Risk Management, Proceedings of the 2005 International Conference on Landslide Risk Management, New York, A.A. Balkema, p.63-98.

173. Nakamura, H., 2004, Field instrumentation and laboratory investigation, in Lacerda, W.A., Erlich, M., Fontoura, S.A.B., and Sayao, A.S.F., eds., Landslides—Evaluation and stabilization, Proceedings of the 9th International Symposium on Landslides, London, A.A. Balkema Publishers, v. 1, p. 541-548.

174. National Research Council, 1991, Mitigating losses from land subsidence in the United States, Washington, D.C., National Academy Press, 58 p.

175. Nichol, D., and Graham, J.R., 2001, Remediation and monitoring of a highway across an active landslide at Trevor, North Wales, Engineering Geology, Volume 59, No. 3-4, p. 337-348.

176. O'Neil, G.D., Simmonds, G.R., Grivas, D.A., and Schultz, B.A., 1996, Rainfall-ground movement modelling for natural gas pipelines through landslide terrain, in Proceedings of the 1st International Pipeline Conference, Calgary, Alberta, September.

177. O'Rourke, T.D., Jezerski, J. M., Olson, N. A., Bonneau, A.L., Palmer, M.C., Stewart, H.E., O'Rourke, M. J., and Abdoun, T., 2008, Geotechnics of pipeline system response to earthquakes, Proceedings of Geotechnical Earthquake Engineering and Soil Dynamics IV, May.

178. Osterkamp, T.E., and Romanovsky, V.E., 1996, Characteristics of changing permafrost temperatures in the Alaskan Arctic, U.S.A., Arctic and Alpine Research 28 (3), p. 267–273.

179. Pack, R.T., Tarboton, D.G., and Goodwin, C.N., 1999, GIS-based landslide susceptibility mapping with SINMAP, Proceedings of the 34th symposium on Engineering Geology and Geotechnical Engineering, in Bay, J.A., ed., p. 210-231.

180. Parson E.A, Carter, L., Anderson, P., Wang, B., and Weller, G., 2001, Potential

consequences of climate variability and change for Alaska, in National Assessment Synthesis Team, Climate change impacts on the United States—The potential consequences of climate variability and change, Cambridge, U.K., Cambridge Univ. Press, Report for the U.S. Global Change Research Program, Chapter 10, p. 283–312, http://www.usgcrp.gov/usgcrp/Library/nationalassessment/10Alaska.pdf accessed April 30, 2007.

181. Paulin, M.J., R. Phillips, J.I. Clark, A. Trigg and I. Konuk, 1998, A full-scale investigation into pipeline/soil interaction, Proceedings of the International Pipeline Conference, Calgary, Alberta, American Society of Mechanical Engineers, p. 779-787.

182. Petley, D.N., 2004, The evolution of slope failures—Mechanisms of rupture propagation, Natural Hazards and Earth System Sciences, v. 4, p. 147-152.

183. Picarelli, L., and Russo, C., 2004, Remarks on the mechanics of slow active landslides and the interction with man-made works, in Lacerda, W.A., Erlich, M., Fontoura, S.A.B., and Sayao, A.S.F., eds., Landslides—Evaluation and stabilization, Proceedings of the 9th International Symposium on Landslides, London, A.A. Balkema Publishers, v. 2, p. 1141-1176.

184. Pieraccini, D., Casagli, N., Luzi, G., Tarchi, D., Mecatti, D., Noferini, L. and Atzeni, C., 2003, Landslide monitoring by ground-based radar interferometry: a field test in Valdarino (Italy), International Journal of Remote Sensing, vol.24, no,6, 1385-1391.

185. Pierson, T.C., 2007, Dating young geomorphic surfaces using age of colonizing Douglas Fir in southwestern Washington and northwestern Oregon, USA, Earth Surface Processes and Landforms, vol. 32, p. 811-831, doi:10.1002/esp.1445.

186. Pierson,T.C., 2005, Distinguishing between debris flows and floods from field evidence in small watersheds, U.S. Geological Survey Fact Sheet 2004-3142.

187. Puebla, H., Butler, R.C., O'Neill, E.S., Williams, R.R., 2006, Static and seismic stress-deformation analyses of a deep soil mix wall, 4th International FLAC Symposium on Numercial Modelling in Geomechanics, Paper 04-05, Itasca Consulting Group, Inc., ISBN 0-9767577-0-2.

188. RADARSAT International, 1995, RADARSAT illuminated, your guide to products and services, RADARSAT International Client Services, Richmond, BC, Canada.

189. Reichenbach, P.; Galli, M.; Cardinalli, M.; Guzzetti, F.; and Ardizzone, F., 2004, in Glade, T. Anderson, M., and Crozier, M.J., eds., Landslide Hazard and Risk, John Wiley and Sons, Ltd., p. 429-468.

190. Reid, M.E., 1997, Slope instability caused by small variations in hydraulic conductivity, Journal of Geotechnical and Geoenvironmental Engineering, v. 123, no. 8, p. 717-725.

191. Reid, M.E., 2003, personal communication.

192. Rizkalla, M., Trigg, A., and Simmonds, G., 1996, Recent advances in the modeling of longitudinal pipeline/soil interaction for cohesive soils, Proceedings of the 15th

International Conference on Offshore Mechanics and Arctic Engineering, American Society of Mechanical Engineers, Vol. V, p. 325-332.

193. Rowlands, K.A., Jones, L.D., Whitworth, M., 2003, Landslide laser scanning: a new look at an old problem, Quarterly Journal of Engineering Geology and Hydrogeology, v. 36, p. 155–157.

194. Saito, M., 1965, Forcasting the time of occurrence of slope failure, Proceedings of the Sixth International Conference of Soil Mechanics and foundation Engineering, Montreal, Canada, v. 2, p. 537-541.

195. Samui, P., and Kumar, B., 2006, Artificial neural network prediction of stability numbers for two-layered slopes with associated flow rule, The Electronic Journal of Geotechnical Engineering, v. 11, bundle A, on-line at http://www.ejge.com/2006/Ppr0626/Ppr0626.htm (accessed 3/2/2007)

196. Sarma, S.K., and Tan, D., 2006, Determination of critical slip surface in slope analysis, Geotechnique, Volume: 56, No.: 8, p.: 539-550, DOI: 10.1680/geot.2006.56.8.539

197. Sassa , K., Jukuoka, H., Wang, G., and Ishikawa, N., 2004, Undrained dynamic-loading ring-shear apparatus and its application to landslide dynamics, Landslides, v. 1, no. 1, p. 7-19.

198. Savage, J.E., Savage, W.Z., and Huntoon, P.W., 2003, Development of deep-seated landslides in the Grand Canyon, Proceedings of the 39th U.S. Rock Mechanics Symposium, v.2, p. 2,471-2,475.

199. Savage, W.Z., Baum, R.L., Morrissey, M.M., and Arndt, B.P., 2000, Finite element analysis of the Woodway Landslide, Washington, U.S. Geological Survey Bulletin 2180, 9p.

200. Schulz, W.H., 2004, Landslide mapping using LIDAR imagery Seattle, Washington, U.S. Geological Survey Open File Report 2004-1396: http://pubs.usgs.gov/of/2004/1396/OF2004-1396_508.pdf

201. Schulz, W.H., 2005, Landslide Susceptibility Estimated From Mapping Using Light Detection and Ranging (LIDAR) Imagery and Historical Landslide Records, Seattle, Washington, U.S. Geological Survey Open File Report 05-1405, http://pubs.usgs.gov/of/2005/1405/

202. Schuster, R. L. and Fleming, R. W., 1986, Economic losses and fatalities due to landslides, Bulletin of the Association of Engineering Geologists, 23(1), 11-28.

203. Schuster, R. L. and Pringle, P. T., 2002, Engineering history and impacts of the Bonneville landslide, Columbia River gorge, Washington - Oregon, USA; in Rybar, J., Stemberk, J.; Wagner, P., eds., Landslides, Proceedings of the first European Conference on Landslides, Prague, Czech Republic July 22-24, 2002, Lisse, Netherlands, A.A. Balkema Publishers.

204. Sharma, S., 2007, Slope stability assessment using limit equilibrium methods, in Turner, A.K., and Schuster, R.L., editors, Landslides and Society, Keynote and invited presentations at the First North America Landslide Conference in Vail,

Colorado, June 3-8, Association of Engineering Geologists Special Publication Number 22, p. 239-260.

205. Sheth, A.N., Tejaswi, K, Mehta, P., Parekh, C., Bansal, R., Merchant, S., Singh, T.N., Desai, U.B., Thekkath, C.A., and Toyama, K., 2005, SenSlide—A sensor network based landslide prediction system [abs.], Proceedings of SenSys 2005, November 2-4, 2005.

206. Shiklomanov, N.I., Anisomov, O.A., Romanovsky, V.E., and Zhang, T., 2004, Spatially distributed permafrost models—Current status, problems and needs, EOS Trans. AGU, 85(47), Fall Meet, Supplemenatal Abstract C12A-01.

207. Skempton, A.W., 1985, Residual strength of clays in landslides, folded strata, and the laboratory, Geotechnique, vol. 35, p. 3-18.

208. Skempton, A.W., Leadbeater, A.D., and Chandler, R.J., 1989, The Mam Tor landslide, north Derbyshire, Philosophical Transactions of the Royal Society of London, v. A329, p. 503-547.

209. Smith, I.M., and Griffiths, D.V., 2004, Programming the finite element method, 4th ed., Wiley, 628 p.

210. Stead, D., Coggan, J.S., and Eberhardt, E., 2004, Modeling of complex rock slope failure mechanisms using a hybrid finite-discrete element code, in Lacerda, W.A., Erlich, M., Fontoura, S.A.B., and Sayao, A.S.F., eds., Landslides—Evaluation and stabilization, Proceedings of the 9th International Symposium on Landslides, London, A.A. Balkema Publishers, v. 2, p. 1067-1072.

211. Stevens, W.D., 1998, ROCFALL --A tool for probabilistic analysis, design of remedial measures and prediction of rockfalls, Toronto, University of Toronto, Master of Applied Science thesis, 28 p., 2 appendices, (http://www.rocscience.com/downloads/rocfall/wds98.zip, accessed August 22, 2007).

212. Stoffel, M., 2006, A review of studies dealing with tree rings and rock fall activity - the role of dendrogeomorphology in natural hazard research, Natural Hazards, v. 51-70, DOI 10.1007/s11069-005-2961-z.

213. Tan, E., Fredlund, D.G., and Marjerison, B., 2007, Installation procedure for thermal conductivity matric suction sensors and analysis of their long-term readings, Canadian Geotechnical Journal 44(2): 113–125, doi:10.1139/T06-098.

214. Tarayre, H., and Massonnet, D., 1996, Atmospheric propagation heterogeneities revealed by ERS-1 interferometry, Geophysical Research Letters, vol. 23, no. 9, p. 989-992, May 1996.

215. Tarchi, D., Antonello, G., Casagli, N., Farina, P., Fortuny-G., Joachim, G., Letizia, L.D., 2005, On the use of ground-based SAR interferometry for slope failure early warning—the Cortenova rockslide (Italy), Chapter 43 in Sassa, K., Fukuoka, H., Wang, F. and Wang, G., editors, Landslides—Risk analysis and sustainable disaster management, Berlin, Springer, p. 337-342.

216. Tarchi, D., Casagli, N., Fanti, R., Leva, D.D., Luzi, G., Pasuto, A., Pieraccini, M.,

and Silvano, S., 2003, Landslide monitoring by using ground-based SAR interferometry—An example of application to the Tessina landslide in Italy, Engineering Geology, v. 68, No. 1-2, p. 15-30.

217. Tolman, C.F., and Poland, J.F., 1940, Ground-water infiltration, and ground-surface recession in Santa Clara Valley, Santa Clara County, California, American Geophysical Union Transactions, v. 21, part 1, p. 23–34.

218. Troncone, A., 2005, Numerical analysis of a landslide in soils with a strained softening behavior, Geotechnique, volume 55, number 8, p. 585-596.

219. Turner, J. E., 2004, Lateral force-displacement behavior of pipes in partially saturated sand, M.A.Sc. Thesis, Cornell University, Ithaca, New York.

220. Turner, A.K., and Schuster, R.L. eds., 1996, Landslides—Investigation and mitigation, Transportation Research Board Special Report 247, 673 p.

221. U.S. Geological Survey, 1996, Permafrost map of Alaska, Eros Alaska field office, map metadata, scale 1:2,500,000, http://agdc.usgs.gov/data/projects/fhm/, accessed May 10, 2007.

222. Van Den Eeckhaut, M., Verstraeten, G., Poesen, J., 2007, Morphology and internal structure of a dormant landslide in a hilly area—the Collinabos landslide (Belgium), Geomorphology, doi:10.1016/j.geomorph.2006.12.005.

223. Van Westen, C.J., 2004, Geo-information tools for landslide risk assessment—an overview of recent developments, in Lacerda, W.A., Erlich, M., Fontoura, S.A.B., and Sayao, A.S.F., eds., Landslides—Evaluation and stabilization, Proceedings of the 9th International Symposium on Landslides, London, A.A. Balkema Publishers, v. 1, p. 759-765.

224. Varnes, D.J., 1978,Slope movement types and processes, Landslides Analysis and Control, Special Report 176, Transportation Research Board, National Academy of Sciences, p. 11-33.

225. Vasconcellos, C.R.A, Freitas, J.C., and Oliveira, H.R., 2004, A historical case in the Bolivia-Brazil natural gas pipeline—slope on the Curriola River, Proceedings of the International Pipeline Conference, American Society of Mechanical Engineers, paper IPC04-0400.

226. Voight, B., 1988, A relation to describe rate-dependent material failure, Science, v. 243, p. 200–203.

227. Werner, C., Wegmuller, U., Strozzi, T. and Wiesmann, A., 2003, Interferometric point target analysis for deformation mapping, IGARSS, Toulouse, France 21-25 July 2003.

228. Wieczorek, G.F., & Glade, T. 2005, Climatic factors influencing occurrence of debris flows, in Jakob, M. and Hungr, O., eds., Debris-flow hazards and related phenomena, Berlin Heidelberg, Praxis Springer, 325-362.

229. Wieczorek, G.F., Morrissey, M.M., Iovine, G., and Godt, J., 1999, Rock-fall potential in the Yosemite Valley, California, U.S. Geological Survey Open File Report 99-578, 1 plate 1:12.000, 7 p. http://greenwood.cr.usgs.gov/pub/open-file-

reports/ofr-99-0578/

230. Wijewickreme, D., Karimian, H., and Honegger, D., 2008, Response of buried steel pipelines subject to relative axial loading, Canadian Geotechnical Journal, in press.

231. Williams, R.A., and Pratt, T.L.1996, Detection of the base of Slumgullion landslide by seismic reflection and refraction methods in Varnes, D.J., and Savage, W.Z., editors, The Slumgullion earth flow—A large-scale natural laboratory, U.S. Geological Survey Bulletin 2130, ch. 13, p. 77-83.

232. Wu, T.H., 1969, Soil mechanics, Allyn and Bacon, Inc., Boston, 431 p.

233. Wu, T. H., 1996, Soil strength properties and their measurement, Chapter 12 in Landslides—Investigations and mitigation, Turner, A.K., and Schuster, R.L. eds., Transportation Research Board, Special Report 247, p. 319-336.

234. Wu, T.H., Tang, W.H., and Einstein, H.H., 1996, Landslide hazard and risk assessment, in Turner, A.K., and Schuster, R.L., eds., Landslides—Investigation and mitigation, Transportation Research Board, National Research Council Special Report 247, p. 106-118.

235. Wyllie, D.C., and Norrish, N.I., 1996, Rock strength properties and their measurement, in Turner, A.K., and Schuster, R.L., eds., Landslides—Investigation and mitigation, Transportation Research Board, National Research Council Special Report 247,p. 372-390.

236. Yimsiri, S., Soga, K., Yoshizaki, K., Dasari, G.R., and O'Rourke, T.D., 2004, Lateral and upward soil-pipeline interactions in sand for deep embedment conditions, Journal of Geotechnical and Geoenvironmental Engineering, American Society of Civil Engineers, vol. 130, no. 8.

237. Yoosef-Ghodsi, N., Kulak, G.L., and Murray, D.W., 1994, Behavior of girth-welded line pipe, Structural Engineering Report 203, Department of Civil Engineering, University of Alberta, Canada.

238. Yoshizaki, K., Ando, H., and Oguchi, N., 1998, Large deformation behavior of pipe bends subject to in-plane bending, Proceedings of the International Pipeline Conference, American Society of Mechanical Engineers, p. 733-740.

239. Yoshizaki, K., O'Rourke, T. D., and Hamada, M., 2001, Large deformation behavior of buried pipelines with sow-angle elbows subjected to permanent ground deformation, Journal of Structural Mechanics and Earthquake Engineering, I-2130, p. 41-52.

240. Zebker, H.A. and Villasenor, J., 1992, Decorrelation in interferometric radar echoes, IEEE Transactions on Geoscience and Remote Sensing, vol. 30, no. 5, p. 950-959, Sept. 1992.

241. Zebker, H.A., and Goldstein, R.M., 1986, Topographic mapping from interferometric synthetic aperture radar observations, Journal of Geophysical Research, vol.91, (B5), p.4993-4999, April 1986.

242. Zhu, D.Y., Lee, C.F., Qian, Q.H., and Chen, G. R., 2005, A concise algorithm for computing the factor of safety using the Morgenstern–Price method, Canadian

Geotechnical Journal 42(1): 272–278, doi:10.1139/t04-072.

243. Zi, G., and Belytschko, T., 2003, New crack-tip elements for XFEM and applications to cohesive cracks, International Journal of Numerical Methods in Engineering, vol. 57, p. 2221-2240.

244. Zimmerman, T.J.E, Stephens,M.J., DeGreer, D.D., and Chen, Q., 1995, Compressive strain limits for buried pipelines, Proceedings of the 1995 Offshore Mechanics and Arctic Engineering Conference, American Society of Mechanical Engineers, Volume V, p. 365-378.