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Deliverable D5.1

Compendium of tested and innovative structural, non-structural and risk-transfer mitigation measures for different landslide types

Work Package 5.1 – Toolbox for landslide hazard and risk mitigation measures

Deliverable/Work Package Leader: SGI/ICG

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SUMMARY

Deliverable D5.1 provides a compendium of tested and innovative structural and non-structural (including risk-transfer) mitigation measures for different landslide types, to be used both as a basis for the web-based "toolbox" and as a resource for a wide variety of users. Emphasis has been placed on providing a rational framework applicable to all the measures listed in the compendium and to any other specific measure that may be developed in the future. In the context of the SAFELAND Project, the classification of mitigation measures has been related to the term of the "risk equation" (hazard, vulnerability, elements at risk) addressed by the specific mitigation measure. The mitigation measures classified here as "stabilization", i.e. reduction of hazard are further subdivided in relation to the triggering factors and mechanisms addressed by each technique.
The text is supplemented by fact sheets that provide specific guidance on hazard mitigation measures, including a brief description, guidance on design, schematic details, practical examples and references. The fact sheets also include a subjective rating of the applicability of the specific mitigation measure in relation to the descriptors used for classifying landslides. These ratings have to be considered indicative only and subject to further refinement.

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Disclaimer

Every effort has been made to ensure that all the information and recommendations in this Compendium are accurate and up to date. However, each landslide is different from all others and technology evolves continuosly. It shall be the responsibility of the users before implementing any mitigation measure to seek expert advice and to satisfy themselves of the adequacy of the proposed measures for the specifics of the landslide under consideration. The Authors accept no liability for any claim that may arise in relation to the content of this report.

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CONTENTS

ANNEXES

1 INTRODUCTION

Within the general framework of the interrelated work packages and deliverables produced for the SAFELAND Project, the objectives of Work Package 5.1 is to identify and to document cost-effective structural and non-structural landslide mitigation options and to produce a web-based "toolbox” of innovative and technically appropriate prevention and mitigation measures, based on technology, experience and expert judgment in Europe and abroad.

In particular, Deliverable D5.1 is intended to provide a compendium of both tested and innovative structural and non-structural (including risk-transfer) mitigation measures for different landslide types.
The Deliverable is intended to be used both as a basis for the web-based “toolbox” described above and as a resource for a wide variety of end users, from politicians and planners who may wish to access and understand the underlying technical information to engineers who may be involved in the “nuts-and-bolts” of implementing mitigation measures for a specific application.

As will be discussed in greater detail below, in general terms, for the purposes of the deliverable,

- “structural” measures include, but are not limited to drainage, erosion protection, channelling, vegetation, ground improvement, barriers such as earth ramparts, walls, artificial elevated land, anchoring systems and retaining structures; buildings designed and/or placed in locations to withstand the impact forces of landslides and to provide safe dwellings for people, and escape routes;
- “non-structural” or more generally “consequence reducing measures” include, but are not limited to: retreat from hazard, land-use planning, early warning, public preparedness, (escape routes, etc.) and emergency management.

Continuous technological progress and innovation make it virtually impossible to provide an exhaustive and detailed list. Each of the techniques or approaches described in this compendium could have many variations, reflecting differences resulting for example from:

- specific conditions which vary form place to place;
- technological development;
- commercial interests to differentiate products to overcome patents and copyright;
- different or changing legislation.

Apparent variations may result also from the use of different terminology to describe substantially the same measure.

While every effort has been made to provide a comprehensive and balanced compendium, inevitably readers will note omissions and, possibly, apparent repetition. Many may be the result of having to apply personal judgement in deciding whether to make a particular distinction or to include reference to proprietary systems; all queries and suggestions will be welcome.

In drafting the compendium, particular emphasis has been placed on providing a rational framework applicable to all the measures listed in the compendium and to any other specific measure that may be developed in the future. In the context of the SAFELAND Project and in light of the general consensus on a risk based approach to landslide management, it is believed that the classification system that best suits the objectives and contents of the Project is to relate the classification of mitigation measures to the term of the “risk equation” which is specifically addressed by the specific mitigation measure.

With regard to technical and practical details, these are necessarily provided only in broad terms. While sufficient details are provided to describe the nature and the specific characteristics of each mitigation measure, with reference to practical examples where possible, it must be clear that it is not within the scope of this document to provide detailed guidance on design and implementation, which should be addressed on a case by case basis by suitably qualified and experienced professionals with reference to the specific regulations applicable from place to place and with local practice.

Reflecting these broad objectives, the structure of the report includes:

- a brief discussion of the classification of the possible mitigation measures detailed in the report;
- guidance on the applicability and effectivness of each mitigation measure considered to different types of landslides;
- information on the maturity of the technology, which can range fom “prototype development” to “obsolete”;
- information on current design methods, their maturity and associated uncertainties;
- comparative (qualitative) information on costs.

For ease of reference, all the information relating to each mitigation measure considered is also summarized in fact sheets, which also include brief descriptions of practical examples and further references.

2 CLASSIFICATION OF MITIGATION MEASURES

It is widely accepted and is the backbone of the SAFELAND Project that the management of landslides and engineered slopes involve some form of risk assessment and risk management. (Ambrozic et al., 2009).

Engineering judgment has been progressively supported by formal application of risk assessment and management principles, initially qualitatively in the 1970's and 1980's, later also quantitatively, starting from the 1990's. These developments are described for example by Varnes (1984), Whitman (1984), Einstein (1988, 1997), Fell (1994), Leroi (1996), Wu et al. (1996), Fell and Hartford (1997), Nadim and Lacasse (1999), Ho et al. (2000), Kvalstad et al. (2001), Nadim et al. (2003), Nadim and Lacasse (2003, 2004), Hartford and Baecher (2004), Lee and Jones (2004), as summarised by Ambrozic et al. (2009).

Figure 1 summarizes the framework for landslide risk management (Fell et al., 2005; Hungr et al., 2005); it is widely used internationally and has been adopted as the reference framework in the "Guidelines for landslide susceptibility, hazard and risk zoning for land use planning" published by Fell et al. (2008) on behalf of the JTC-1 Joint Technical Committee on Landslides and Engineered Slopes.

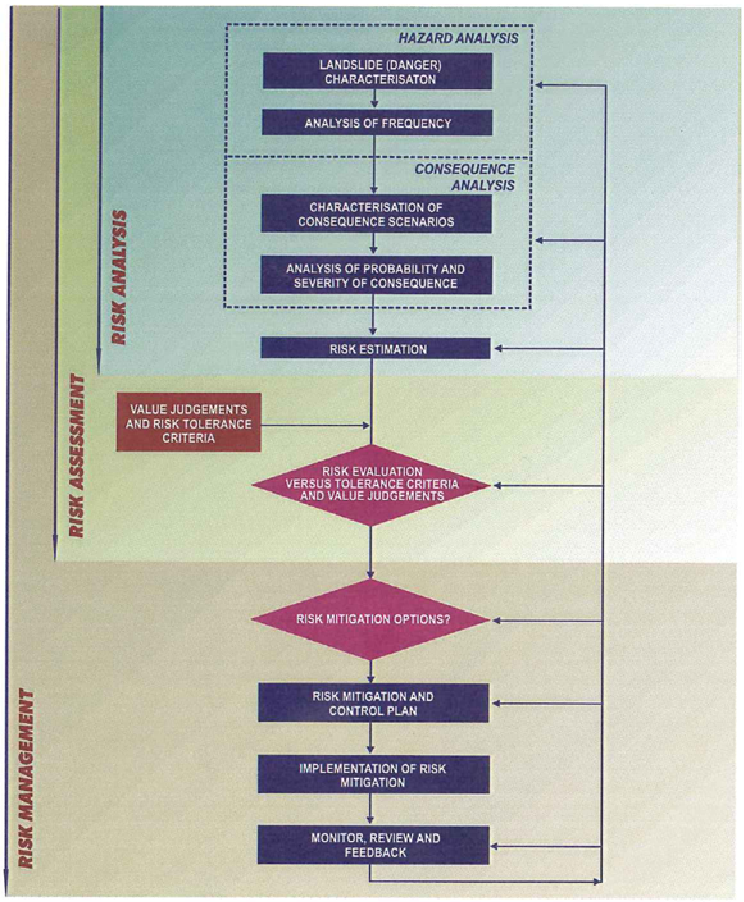


Figure 1: Framework for landslide risk management (after Fell et al., 2008)

As shown in **Figure 1**, the evaluation, implementation and control of mitigation measures fall within this framework and in fact complete and complement the risk analysis and risk assessment stages of the process and it is therefore useful to relate the classification of mitigation measures to the same principles and criteria used in the rest of the process.

The principles, current practice, prospected development and example application of the Quantitative Risk Assessment (QRA) of landslides is addresses in detail in other Work Packages of the SAFELAND Project and reference should be made to the appropriate deliverables, especially:

- D2.8 "Recommended procedures for validating landslide hazard and risk models and maps";
- D2.9 "Toolbox for landslide quantitative risk assessment";
- D2.10 "Identification of landslide hazard and risk hotspots in Europe"; and
- D2.11 "QRA case studies at selected hotspots".

Only the basic principles are referred to here, as they provide the backdrop for the proposed classification of mitigation measures.

Notwithstanding the significant efforts spent in attempting to attain a unified set of definitions and terminology, some variations remain in the literature. For the avoidance of doubt, the terms used in this report are defined below based on the internationally accepted definitions provided by the "Guidelines for landslide susceptibility, hazard and risk zoning for land use planning" (Fell et al., 2008), which have also been adopted for the SAFELAND Project (See Project Glossary in Deliverable D8.1 for full list):

- *Hazard (H_i)* means the probability of occurrance within a specified period of time and within a given area of a specific (i^{th}) potentially damaging phenomenon occurring in or otherwise impinging on the area.
- *Vulnerability (V_i)* means the degree of loss to a given element or set of elements at risk (see below) resulting from the occurrence of a specific (i^{th}) phenomenon of a given magnitude impinging on the area.
- *Elements at risk (E)* means the population, buildings, engineering works, economic activities, public services utilities, other infrastructure and environmental values in a given area.
- *Total Risk (R_{ti})* means a measure of the probability and severity of an adverse effect to health, property or the environment or disruption of economic activity due to a specific (i^{th}) phenomenon.

The Total Risk R_{ti} due to a particular (i^{th}) phenomenon within a specified period of time and within a given area can be expressed as:

$$R_{ti} = (E) \cdot (H_i \cdot V_i) \qquad [1]$$

It should be noted that the definition of "Elements at risk" does not include only an "inventory" of the number and type of elements exposed but also some measure of their "value", whereby reference is sometimes made to definitions which differenciate between numbers, specific value and overall value. While these distinctions and possible refinements and the practical and ethical problems they pose may be of interest for QRA, they are not essential for the purposes of this report and are not discussed further here.

The Total Risk R_t from all (N) possible landslide phenomena within a specified period of time and within a given area is the sum of the risk posed by all the specific (i^{th}) phenomena that impinge on the area of interest, subject to considerations of conditional probabilities of occurrence and to "domino chains", i.e. the progressive triggering of distinct phenomena in a linked sequence of cause and effect (e.g. large landslide → natural dam → overttoping → debris flow etc.).

$$R_t = \sum_{i=0}^{N} (E_i) \cdot (H_i \cdot V_i) \qquad [2]$$

It is evident that the Total Risk can be mitigated by reducing (see for example Canuti and Casagli, 1994):

- the Hazard (i.e. the probability of occurrence of one or more phenomena);
- the Vulnerability (i.e. the the degree of loss to the elements at risk for a given hazard);
- the Elements at risk (i.e. their number and/or specific value).

This represents a useful basis for classifying mitigation measures, because it provides a direct link with QRA and it highlights where the benefits of the mitigation measure being considered are accrued.

Other classifications of mitigation measures have been proposed, based on similar concepts but expressed in different terms. For example, Evangelista et al. (2008) distinguish between:

- *Stabilization*: measures which increase the "margin of safety" of the slope or that intercept the run out (structural measures);
- *Restrictions on the use of the element at risk*: permanently or temporarily;
- *Restrictions on land usage:* through [land use planning tools], to limit the presence of elements at risk in the area threatened by the landslide (non-structural measures);
- *Actions by the Civil Protection authorities:* which allow to remove from the area threatened by the landslide within a suitably short reaction time most valuable elements at risk, including as a minimum human life (emergency plans).

In partial analogy with the title of this report, Evangelista et al. (2008) use the terms "structural" and "non structural", although they apply the terms to cover only part of the full range of possible mitigation measures.

Similarly, Ambrozic et al. (2009) identify the following possible strategies for risk management:

- *Avoidance*: can be implemented at the land-use planning stage for proposed development sand/or to relocate existing facilities, if possible;
- *Tolerance:* can be implemented if the risk level is deemed to be sufficiently low such that direct or indirect costs associated with other strategies cannot be warranted.

Possible actions include “do nothing” or risk reallocation through private insurance or explicit or implicit promises of public intervention such as declaration of a “state of emergency” and the awarding of special funding and compensation to victims;

- *Monitoring/warning*: can be implemented when landslide hazards affect large territories or when dealing with massive potential landslides. It provides additional information to enhance risk assessment and allows the implementation of warning systems for the temporary evacuation of the population at risk;
- *Stabilization*: requires the implementation of engineering works to reduce the probability of occurrence of landslides;
- *Control works*: requires the implementation of engineering works to protect/reinforce/isolate the elements at risk from the influence of landsliding

Ambrozic et al. (2009) also refer more generally to:

- Measures to reduce the hazard (through reducing the probability of triggering through stabilization and/or by reducing subsequent ground movement through barriers or containment);
- Measures to reduce the vulnerability (i.e. reducing the consequences of failure).

This last statement exemplifies some of the difficulties that arise in classifying mitigation measures. In particular:

- although it may be justified in some respects to classify barriers and containment as hazard reducing measures, in the context of area wide risk management they might be better classified as measures to reduce the exposure of the elements they protect;
- avoidance may be as effective at reducing the consequences of failure as reductions in vulnerability, so inferring an exclusive association between reducing vulnerability and reducing the consequences of failure can be misleading.

These apparent contradictions derive from the definition of “vulnerability”, which Ambrozic et al. (2009) extend to include not only the damage functions with respect to ground movement (vulnerability s.s.), but also the number of the vulnerable elements potentially affected by a landslide and the probability that they will intersect the landslide ground movement.

Similarly, warning/alarm systems associated with plans for emergency evacuation or safe sheltering are often classified as measures to reduce vulnerability. However, keeping to the distinct definitions of “vulnerability” and “elements at risk”, these systems are best classified as measures to reduce (temporarily and selectively) the elements at risk, rather than their vulnerability.

Although they present some significant differences, all the classifications described above are somehow related, having as a common thread some more or less explicit relationship with the constitutive equation of risk. In an attempt to reconcile to a common framework the different terminology used by various authors, **Table 1** summarizes the classification proposed here.

Within the general domain of the mitigation measures classified here as “stabilization”, i.e. reduction of hazard, it is possible to consider a further subdivision in relation to the triggering factors and mechanisms that each technique addresses, as discussed in greater detail in Chapter 4.

Other somehow related, widely used, classifications of stabilization measures include distinctions between:

- “active” and “passive” stabilization measures (Picarelli and Urcioli, 2006; Evangelista et al., 2008), in relation to whether the mitigation measures “actively” pursue an improvement s.s. of the stability of slope, or they “passively” intercept the run out when movement actually occurs, protecting the elements at risk.
- “hard” and “soft” stabilization measures (Parry et al., 2003a, b), where “hard” is normally used to describe structural techniques that are visually obvious, while “soft” is normally used to describe techniques that are visually less intrusive and which improve the strength or other properties of the ground, such as its drainage capability. The terms “hard” and “soft” can also be used in relation to the relative stiffness of the stabilization works and the surrounding soil, which results in the overall behaviour of the stabilized slope being modelled as an equivalent contiuum or as distinct materials. The terms “hard” and “soft” can also be used in direct analogy with the terms “structural” and “non structural”, with the same meaning of hardware and software, depending on whether the mitigation measure addresses tangible, material or intangible, “immaterial” aspects of the risk.
- “preventive” and “remedial” stabilization measures (Parry et al., 2003a, b), relating to their relevance to different stages of movement (see Leroueil, 2001).

Table 1: General classification of mitigation measures

Classification		Component of risk addressed	Brief description	Notes and other terms used
STRUCTURAL	Stabilization	Hazard (H)	engineering works to reduce the probability of occurrence of landsliding	Preventive, remedial, hard, soft, active stabilization
	Control	Vulnerability (V)	engineering works to protect, reinforce, isolate the elements at risk from the influence of landsliding	Preventive, hard, soft, passive stabilization
NON STRUCTURAL	Avoidance	Elements (E)	temporary and/or permanent reduction of exposure through: warning systems and emergency evacuation or safe sheltering, land-use planning and/or relocation of existing facilities	Direct temporary and/or permanent reduction of the number and/or value of elements at risk. Monitoring and warning or alarm systems and associated civil protection procedures, often described as reducing vulnerability, in actual fact operate through temporary, selective avoidance.
	Tolerance	Elements (E)	Awareness, acceptance and/or sharing of risk	Indirect reduction of the number and/or value of elements at risk

3 CRITERIA FOR SELECTION

The selection of the most appropriate mitigation measures to be adopted in specific situations must take into account the following aspects:

- factors which determine the hazard, in terms of the type, rate, depth and the probability of occurrence of the movement or landslide, such as, for example:
    - the physical characteristics of the geosystem, including the stratigraphy and the mechanical characteristics of the materials, the hydrological (surface water) and the hydrogeological (groundwater) regime;
    - the morphology of the area;
    - the actual or potential causative processes affecting the geosystem, which can determine the occurrence of movement or landslides;
- factors which affect the nature and quantification of risk for a given hazard, such as the presence and vulnerability of elements at risk, both in the potentially unstable area and in areas which may be affected by the run-out;
- factors which affect the actual feasibility of specific mitigation measures, such as, for example:
    - the phase and rate of movement at the time of implementation;
    - the morphology of the area in relation to accessibility and safety of workers and the public;
    - environmental constraints, such as the impact on the archeological, hystorical and visual/landscape value of the locale;
    - preexisting structures and infrastructure that may be affected, directly or indirectly;
    - capital and operating cost, including maintenance.

4 MEASURES TO REDUCE HAZARD

4.1 CLASSIFICATION

Mitigation measures which aim to reduce the hazard must reduce the probability of triggering of the landslide(s) which the specific measure is intended to address. This type of mitigation measures are sometimes referred to as "stabilization".

As discussed in Deliverable D1.1 on landslide triggering, independently of the causative processes and the complexity of the specific geosystem under consideration the factors which determine the triggering of movements are:

a) decrease in shear strength $\sum\tau_r$
b) increase in driving shear stress $\sum\tau_d$

The most common causative processes are listed in **Table 2** (adapted from Leroueil, 2001). Combinations of (a) and (b) often act simultaneously as a direct result of external processes, as in the case of basal erosion or excavations, which can cause both an increase in τ_d, through increased slope angle and/or height, or a decrease in τ_r, through a reduction in total and effective stress.

Table 2: Triggering factors with examples of common causative processes (adapted from Leroueil, 2001)

Triggering factor	**Common causative processes**
Decrease in shear strength τ_r	- Infiltration due to rainfall, snowmelt, irrigation, leakage from utilities - Construction activities, e.g. pile diving - Weathering (rebound/swelling, physical, chemical) - Fatigue and excess pore pressure due to cyclic loading
Increase in driving shear stress τ_d	- Erosion or excavation at the toe - Surcharging at the top - Rapid drawdown - Fall of rock onto the slope and other impulsive loading - Earthquake
Note: Many processes affect both τ_d and τ_r; association to one or the other in the table is indicative only	

In order to reduce the probability of triggering, mitigation measures which aim to reduce the hazard of landslides occurring must act in the system in the opposite direction, by:

A increasing the resisting forces; and/or
B decreasing the driving forces.

While this could provide a first step in the classification of this type of mitigation measures, it is more convenient to classify them on the basis of the physical process involved. In particular, it is here recommended to distinguish between the classes indicated in **Table 3**.

Table 3: Landslide Hazard Mitigation Measures
(adapted from Popescu & Sasahara, 2009)

Physical process	**Brief description**
Surface protection; control of surface erosion	• Vegetation (hydroseeding, turfing, trees/bushes) • Fascines/brush. • Geosynthetics. • Substitution; drainage blanket • beach replenishment; rip-rap. • Dentition
Modifying the geometry and/or mass distribution	• Removal of material from the area driving the landslide (with possible substitution by lightweight fill). • Addition of material to the area maintaining stability, with or without gravity, catilever, crib/cellular and/or reinforced soil walls. • Reduction of the general slope angle. • Scaling (removal of loose/unstable blocks/boulders).
Modifying surface water regime – surface drainage	• Diversion channels • Check dams • Surface drains (ditches, piping) to divert water from flowing onto the slide area. • Sealing tension cracks. • Impermeabilization. (*) • Vegetation. (*) Note (*): associated with control of surface erosion
Modifying groundwater regime – deep drainage	• Shallow or deep trenches filled with coarse grained free-draining geomaterials and geosynthetics • Subhorizontal drains • Vertical small diameter wells; self draining (where they provide relief to artesian pressures or underdrainage to a perched acquifer) or drained by siphoning, electropneumatic or electromechanical pumps • Vertical medium diameter wells with gravity drainage through a base collector • Caissons (large diameter wells), with or without secondary subhorizontal drains and gravity drainage • Drainage tunnels, galleries, adits, with or without secondary subhorizontal or subvertical drains and/or as gravity outlet for wells drilled from the surface
Modifying the mechanical characteristics of the unstable mass	• Substitution • Compaction • Deep mixing with lime and/or cement • Permeation or pressure grouting with cementitiuous or chemical binders • Jet grouting • Modification of the groundwater chemistry
Transfer of loads to more competent strata	• Shear keys: counterforts, piles; barrettes (diaphragm walls); caissons • Anchors: soil nails; dowels, rock bolts; multistrand anchors (with or without facing consisting of plates, nets, reinforced shotcrete) • Anchored walls (combination of anchors and shear keys)

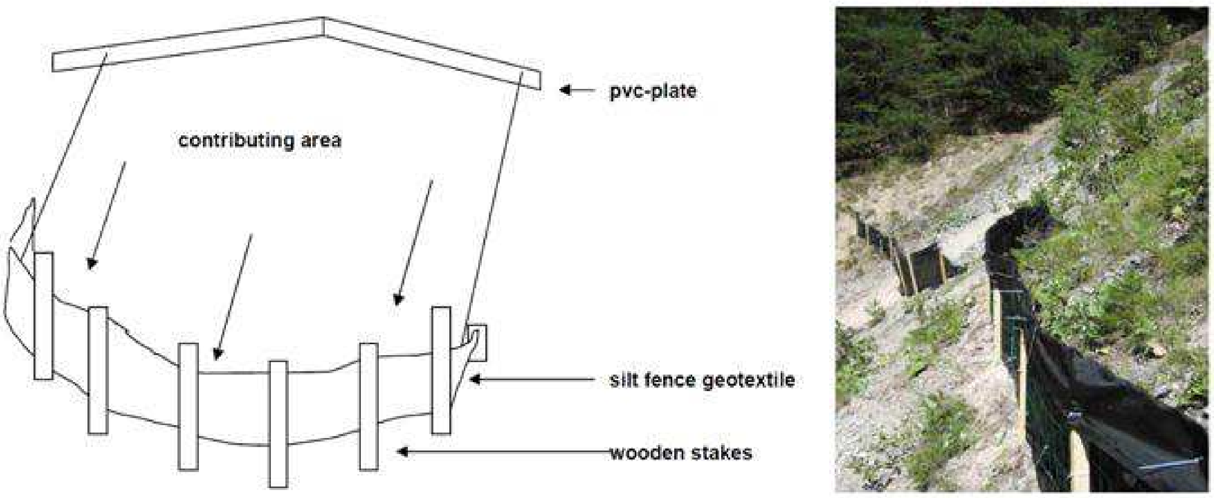
Retaining structures are used extensively and can be considered as an additional class of hazard mitigation measures, even though they are used as means to modify slope geometry and/or to transfer load to more competent strata, rather than to address a specific physical process.

The various techniques available to mitigate landslide hazard are described briefly below and in more detail in the fact-sheets in Appendix A.

4.2 SURFACE PROTECTION AND EROSION CONTROL

Erosion is the displacement of solids (soil, rock) at the ground surface in response to applied by external agents such as wind, water, ice, pedestrian or animal passage.

Various techniques are available to measure soil erosion, including rainfall simulation, erosion bridges, Gerlach troughs and small watershed techniques. They are often costly and time consuming and are not always in widespread use. Therefore, Dissmeyer (1982) developed a protocol to measure hillslope erosion, using silt fences consisting of a synthetic geotextile fabric that is woven to provide structural integrity and small openings that pass water but not coarse sediment. They have low permeability, which make them suitable to form temporary detention storage areas, allowing sediment to settle and water to pass through slowly. Silt fences can be primarily used to compare erosion rates of naturally occurring erosion. Furthermore, the effect of vegetative or mechanical rehabilitation treatment can be investigated. This technique has been applied to the Illgraben catchment (9.5 km^2), situated neat Susten (Leukerbad) in canton Valais, Switzerland. The catchment is characterized by a very high degree of sediment transport activity and shows rapid dynamic landscape changes and evidence of significant erosion events, including frequent large debris flows (Gwerder 2007).



(a) (b)

Figure 2: Silt fence geotextile mitigation measure: (a) schematic representation; (b) application to Illgraben catchemnt (Gwerder 2007)

Within the framework of mitigating landslide hazard, possible techniques to control surface erosion include:

- Vegetation (hydroseeding, turfing, trees/bushes)
- Fascines/brush.
- Geosynthetics.
- Substitution with drainage blanket
- Dentition consists of masonry or stone pitching or concrete protection to localized soft/erodible material in a rock face. A grout pipe may be provided for subsequent grouting to ensure good contact between the overhang and the supporting concrete

With particular reference to the use of bio-engineering systems, the main goal of erosion control is to protect the face of the slope and to strengthen subsurface parts, typically by interlocking soil particles with a complex matrix of roots. The stability of slopes is dependent on the ratio of driving forces and the strength of the soil-root system. The weight of vegetation growing on the slope accounts for a part of the driving forces but the roots add to the shear strength of the soil. Vegetation also intercepts rain, by reducing its impact energy and preventing splash erosion and slowing down runoff.

Vegetation also changes the pore pressure in the soil via the evapotranspiration process (Morgan & Rickson, 1995). This process decreases the pore pressure and increases the effective stresses in the soil, which also improves the shear strength (**Figure 3**). But unfortunately, in temperate European climates, the season of peak water demand by vegetation (summer) is out of phase with the season of greatest rainfall (winter) (Smethurs et al., 2006 and Thielen et al., 2011).

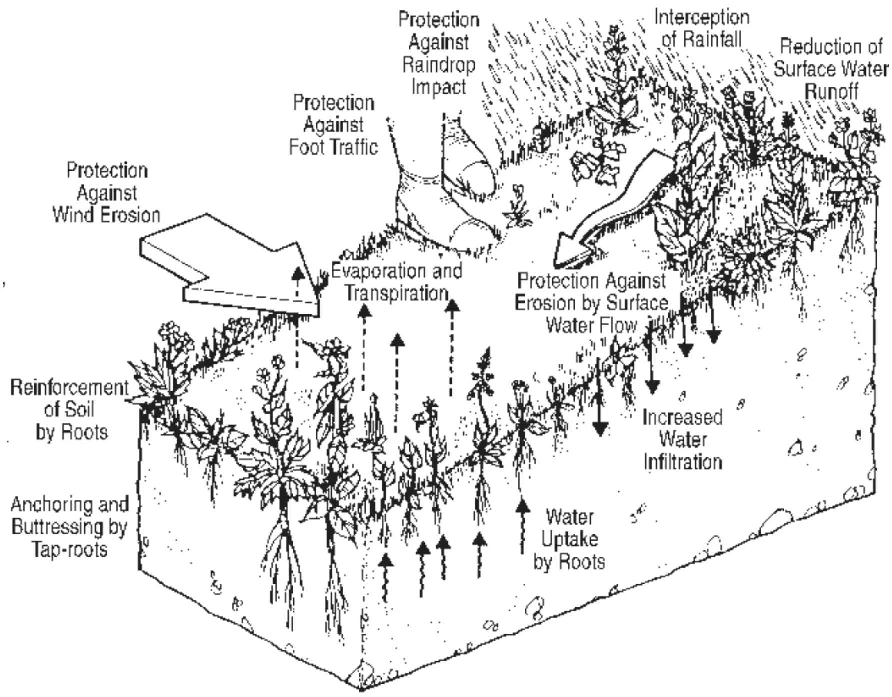


Figure 3: Some influences of vegetation on the soil. (Coppin & Richards, 1990)

Initial conditions for bio-engineering measures are usually rather unfavourable. The area to be stabilised is often barren, partly unstable and erosive processes abound (Graf & Gerber 1997; Graf et al. 2003).

Bio-engineering systems are usually established by conventional seeding of the plants or live planting (Morgan & Rickson, 1995). The main goal of these systems are reducing surface erosion and reinforcing the soil. The construction methods used mainly rooted cuttings and these are installed in different configurations. The effectiveness of this system as soil reinforcement depends on the depth at which cuttings can be placed and the depth to which the roots can penetrate. Soil reinforcement systems by bushes and trees are described by Gray& Leiser (1982), Copping & Richards (1990). The growth rate of roots is related to the volume of the cuttings and some guides on choice and preparation of cuttings have been given by Gray & Leiser (1982) and Schiechtl (1980). For stability, the species should have a root system that penetrates to the required depth. In humid regions, bushes and trees with high transpiration would be more effective in decreasing soil moisture.

Wherever feasible, native vegetation is preferred and the succession from pioneer to climax bush or tree in the site environment, primarily climate and soil type and moisture, should be considered (Morgan & Rickson, 1995, Gray & Leiser 1982, Schiechtl 1980).

The long-term effects of bio-engineering stabilisation methods depend on site characteristics, slope failure processes and the technical and biological measures employed (Stokes et al. 2007). Detailed analysis of the stability of the slope is necessary to determine the suitable stabilising method. One of the greatest uncertainities concerns the depth of the potential sliding surface and the measures have to be chosen accordingly.

Slope stability and the efficiency of stabilising measures are usually influenced not only by soil mechanics but also by hydrological factors and hydraulics. The combined effects are rather complex and are often responsible for failure (Boll, 1997). Surface erosion and landslides are usually long-term processes (over some decades and more) and stabilising measures are required to have a correspondingly long lifespan. The bearing capacity and functionality of supporting structures are likely to become critical in the course of time, and biological measures may fail to prosper. Periodical site inspections are therefore necessary to plan maintenance and/or replacements properly. Knowledge about the development and long-term behaviour of joint technical and biological methods is indispensible (Pastorok et al., 1997; Anand & Desrochers, 2004).

In recent years, several studies have been performed to describe vegetation effects quantitatively. According to Simon & Collison (2002), root-permeated soil makes up a *composite* material that has an enhanced strength. In general, soil can resist against compression stress, but can hardly resist against tensile stress. The fibrous roots of trees and herbaceous plants, on the other hand, can resist against tensile stress, but hardly against compression stress (Nilaweera & Nutalaya, 1999). However, to implement this analysis method in practice, there are restrictions with respect to the root distribution. Usually, only man-made brush layers achieve this condition. Therefore, this model is inappropriate to provide a generalised representation of vegetation effects (Frei, 2009).

If a slip plane is penetrated by roots, they can be included in stability analyses comparable to ground anchors operating with a tieback function. But it requires careful attention to determining the exact root distribution, as well as the pull-out resistance of the different root classes to be able to quantify any anchoring effect of roots. Therefore, this model is inappropriate.

A further possibility is to assign vegetation effects to the soil shear strength directly. In doing so, two approaches can be taken: those that immediately measure the shear strength and methods that assign vegetation effects to the shear strength parameters. The direct measurement of the shear strength of root permeated soils can be performed by means of a direct shear apparatus, as described in Waldron et al. (1983), Wu (1984) and Tobias (1992). According to Boll & Graf (2001), the disadvantage of this method is that the failure plane is predefined (by the apparatus) and that the result obtained by such field tests represents only a pure shear resistance (analogous to a ring-shear test to determine the undrained shear strength of a fine grained soil). The influence of shear pane undulation or any other layering or discountinuities my not be taken into account. As a consequence, such a value is not usually appropriate for classical stability analyses. If the shear strength is written according to the Mohr-Coulomb failure criterion (Terzaghi & Peck 1967), then it can be directly integrated in stability analyses. Wu et al. (1979) as well as Wu (1984) assign any vegetation effects to the soil cohesion, by introducing an additional cohesion component due to the root reinforcement (c_r). Variations in mechanical reinforcement at the root-zone-scale are particularly important for small and shallow landslides with areas of 10 to 2000 m^2 (Reneau & Dietrich, 1987). Moreover, complexities arising from the distribution of root sizes and details of root-soil mechanical reinforcement also demonstrate that application of a uniform cohesion term may represent an oversimplified picture that could overlook susceptibilities emerging when a more complete stress–strain relationship of root systems and characteristics of their distribution are included in calculations of slope stability (Schwartz et al., 2010).

Boll & Graf (2001) regard this additional parameter as simple to determine, but it represents the conditions in superficial soil layers far less optimally than the stress-dependent expression in the frictional component of the Mohr-Coulomb notation. Since the roots exert a form of prestress on the surrounding soil grains, this is analogous to increasing the contact stresses which will contribute to additional shear strength through the modification of frictional resistance. Therefore, adding an additional component to the friction angle would represent the mobilised shear resistance under a greater range of valid stress conditions near the surface. It was postulated that it would be more convenient for designers to describe the resistance mobilised and hence the stability in the vegetation influenced superficial soil area. However, there are no suitable models available yet (Frei, 2009).

Schwartz et al. (2010) reviewed the primary geometrical and mechanical properties of root systems and their function in stabilizing the soil mass. They considered the stress–strain relationships for a bundle of roots using the formalism of the fibre bundle model (FBM) that clumps the effects of roots together and offers a natural means for upscaling mechanical behaviour of root systems. They proposed an extension of the FBM, considering key root and soil parameters such as root diameter distribution, tortuosity, soil type, soil moisture and friction between soil and root surface. The spatial distribution of root mechanical reinforcement around a single tree is computed from root diameter and density distributions and is based properties that can be measured easily. The distribution of root reinforcement for a stand of trees was obtained from spatial and mechanical superposition of individual tree values with respect to their positions on a hillslope. This method has been applied to a full scale rainfall triggering test (Springman et al., 2010) and the results of simulated failure zone (Schwartz, 2010) shows good agreemets with the real failure wedge (Askarinejad et al., 2010).

4.3 MODIFYING THE GEOMETRY OR THE MASS DISTRIBUTION

Total or partial removal of the actually or potentially unstable mass, toe weighting and more generally modification to the geometry and/or mass distribution of slopes are widely used techniques to mitigate the hazard, and to some extent the consequences, of landsliding. Possible modifications to the geometry of the slope include:

- Total removal by mass excavation of the actually or potentially unstable soil and/or rock mass; a special case is representated by trimming and scaling to remove individual hoverhangs, bulges or loose blocks which pose a rockfall hazard on otherwise stable rock slopes.
- Partial removal by mass excavation of soil and/or rock from the driving area (or more in general, regrading or flattening slope angle) to reduce the driving forces, thereby improving overall slope stability.
- Where necessary, for example to preserve the integrity of infrastructure, the excavated mass may be substituted, in whole or in part, by lightweight fill using naturally occurring (geological) materials such as pumice or shells, manufactured materials, such as expanded clay, polystyrene slabs, cellular concrete, and waste materials or byproducts, such as soil mixed with shredded tyres (‘pneusol’), pulverized fly ash, slag, woodchips or logging slash. Lightweight fill is also used to minimize the extent and cost of other mitigation measures by minimizing the adverse effect of construction, for example where alignment constraints may dictate that fills for a new highway be placed in a potentially destabilizing position across an actual or potential landslide.
- Addition of material to the toe or resisting area (or more in general, buttressing, counterweight fills and toe berms), which operates by increasing the resisting forces, thereby improving overall slope stability, by providing sufficient dead weight or restraint near the toe of the unstable slope.

The principles underlaying the complete removal of the potentially or actually unstable mass, be it in soil or rock, including “scaling” otherwise stable rock slopes to remove rockfall hazard, are self explanatory.

Reprofiling, unloading by excavation or by partial replacement with lightweight fill at the head and loading at the toe with fill and/or gravity structures operate on the principle of modifying the balance between driving and resisting forces.

This technique is potentially effective in all materials, except those susceptible to weakening instability or liquefaction. As also summarized for example by Hutchinson (1977), cuts and fills appear to be most effective as a hazard mitigation measure when applied to deep-seated landslides, where the slip surface tends to fall steeply at the head and rise appreciably in the region of the toe (rotational and pseudo-rotational slides). Clearly, the effect of a given cut or fill on the overall factor of safety depends on the size of the landslide being treated.

The correct positioning of cuts and fills on slopes is a great importance, as is proper drainage. The respective merits of removing the head of an actual or potential slide, flattening the slope uniformly or benching it, or of building a berm at its toe have been discussed extensively in the literature.

While localized mitigation by cuts and fills may prove very effective in dealing with the specific failure surface for which they have been designed, it is important to ensure that they do not cause instability themselves, either locally or to the rest of the slope outside the original landslide being addressed. It is important to note also that in some cases, especially in long translational slides, they may be quite ineffective against almost equally serious landslides involving only a portion of the slide, as shown for example slide a-b-d overriding the fill placed to stabilize the slide a-b-c in **Figure 4** (Hutchinson, 1977).

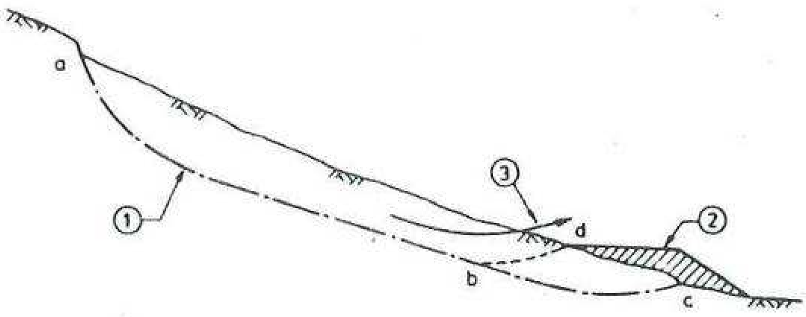


Figure 4: Translational slide stabilized by toe fill and the danger of potential over-rider slides. 1) slip surface; 2) toe fill; 3) over-rider slide (after Hutchuinson, 1977)

4.4 MODIFYING THE SURFACE WATER REGIME

Within the framework of mitigating landslide hazard, possible techniques to modify the surface water regime and their application include:

<u>Major hydraulic works</u>

- Diversion channels, to divert water courses from the toe of the landslide, either to prevent or remediate toe erosion, or to make space for the implementation of other mitigation measures, as was carried out for example on the Taren landslide (Kelly and Martin, 1985). Divesion channels (above ground or in tunnel) are also used to remediate landslide dams, either after the event, as for the Val Pola, Italy 1987 landslide or as a preventive measure, as carried out for the Séchilienne Landslide in France (Durville et al., 2004).

- Check dams, to regulate water courses at the toe of the landslide, to prevent or remediate erosion of the streambed and/or of the banks. Typically, check dams are constructed just downstream of critical areas. However, since they retain sediment, they tend to accelerate erosion further downstream. It is therefore necessary to consider the overall effect on the watercourse as a whole.

Measures to minimize the quantity of surface water flowing into actually or potentially unstable slopes

- Surface drainage works, consisting of ditches, channels, pipework, chutes etc. to collect and direct surface run-off in a controlled manner, to minimize the quantity of surface water flowing into actually or potentially unstable slopes. Ditches and channels should be lined to minimize erosion and uncontrolled infiltration; flexible, self-healing lining or pipes should be used in areas susceptible to cracking and movement. Techniques must be adapted to ground conditions and local technology, favoring adoption; an example of this is provided by Anderson and Holcombe (2004; 2008) who describe the development and application at community level of good drainage practices with locally available, affordable technologies in St. Lucia consisting of ditches lined with a specialised plastic, held in place by a wire mesh (**Figure 5**).

Figure 5: STAR^{TM} drainage system installed by residents in St Lucia, West Indies (after Anderson and Holcombe, 2008)

Measures to minimize the residual amount of surface water flowing into or over actually or potentially unstable slopes actually infiltrating into the ground

- Regrading to facilitate surface run-off, preventing ponding in backtilted areas and grabens caused by previous rotational landsliding.
- Sealing of tension cracks, typically with puddle clay or other impervious fill. It is often sufficient to excavate a trench along the tension crack and to backfill it with the excavated material, possibly adding an impervious membrane near the surface and shaping the ground so that surface water does not pond in the area.
- Covering unprotected slopes with impervious membranes or facing. Impervious membranes are normally used as a short term, temporary or emergency measure, while impervious facing is normally used as a permanent measure on excavated slopes.

Using vegetation to reduce the amount of rainfall reaching the ground and to remove groundwater by evapotraspiration, inducing suction.

The main effect of these measures is to prevent adverse metereological conditions, such as intense and/or prolonged rainfall, snowmelt etc., causing significant adverse variations in the degree of saturation of the aerated zone with the resulting loss of suction and/or variations the piezometric levels, which would result in a reduction of the shear resistance of the ground.
Typically, measures based on the use of vegetation or impermeabilization are also effective in controlling surface erosion and providing local superficial reinforcement of the soil.

4.5 MODIFYING THE GROUNDWATER REGIME

Within the framework of mitigating landslide hazard, possible techniques to modify the surface water regime and their application include:

- Shallow or deep trenches filled with coarse grained free-draining geomaterials and geosynthetics. Trench drains may be located transverly across the top of the slope to intercept groundwater flowing towards the landslide, or within the landslide itself, generally as a series of parallel straight or Y-shaped trenches. Perforated pipes are often placed at the bottom of the trenches to collect water; a geotextile filter fabric is used over the pipe or between the soil and the gravel backfill to prevent occlusions of the drain, preserving the functionality of the trenches in the long term.
- Subhorizontal drains, consisting of perforated pipes ancapsulated in a geotextile filter fabric, if required, and installed in predrilled holes; advances in directional drilling technology allow installation of much longer drains than with conventional drilling and the use of curved profiles to intercept stratified soils. An experimental application of drains installed by directional drilling in the stratified soils at the coastal landslide at Barton-on-Sea, UK in the early 2000’s gave very good results.
- Vertical small diameter wells; self draining (where they provide relief to artesian pressures or underdrainage to a perched acquifer) or drained by siphoning, electropneumatic or electromechanical pumps. The actual method of pumping is selected to suit local conditions. Where applicable, of particular interest for long term applications is the use of siphon wells (Gress, 1996; Bomont, 2004), which minimizes energy consumption.
- Vertical medium diameter wells with gravity drainage through a base collector. The wells are constructed by piling equipment at relatively close spacing along

predetermined alignments both transversal to and along the slope; they are connected at the base by a collector drain which was constructed by drilling from one well to the next by hand held equipment. Recent advances in directional drilling techniques allow the base collector to be drilled without entering the wells, improving safety of installation.

- Caissons (large diameter wells), typically ranging in diameter between 6 and 15 m with or without secondary subhorizontal drains and gravity drainage. Depending on anticipated ground and groundwater conditions, the most common techniques used to form the annular structure are progressive construction during excavation by alternate excavation and casting of consecutive concrete rings or, more frequently, by means of micropiles or piles supplemented by annular steel or concrete ribs installed as excavation proceeds, in which case vertical draining mats are installed in contact with the ground between the piles before casting the final structure, to supplement the drainage provided by the sub-horizontal drains.
- Drainage tunnels, galleries, adits, with or without secondary subhorizontal or subvertical drains and/or as gravity outlet for wells drilled from the surface. Several drainage adits have been constructed to stabilize landslides encountered during the construction of the A1 motorway in Italy in the 1960's. More recent examples are provided by the stabilization works for the Taren Landslide in South wales, UK (Kelly and Martin, 1985) and in the stabilization of the Tablachaca Dam Landslide, Peru (Millet et al., 1992).

All these measures operate by modifying the groundwater regime in such manner as to achieve the following objectives:

- reduce the baseline piezometric level(s) in the slope, including increasing suctions in the aerated zone;
- prevent significant temporary adverse variations of the (reduced) piezometric levels in the slope following adverse metereological events such as intense and/or prolonged rainfall, snowmelt, etc., also preventing temporary saturation and associated loss of suction in the aerated zone from rising groundwater levels.

4.6 MODIFYING THE CHARACTERISTICS OF THE GROUND

Within the framework of mitigating landslide hazard, possible techniques to modify the surface water regime and their application include:

- Substitution; excavation and replacement of unstable mass with other material with improved mechanical characteristics; effective only if it extends to sufficient depth to include the basal failure surface. Normally used for very small landslides only. The use of lightweight fill is discussed at point 4.3.
- Compaction; only effective in granular soils and typically appropriate to reduce the hazard of seismically induced liquefaction and lateral spreading. In applications where there is the possibility of static liquefaction, it must be carried out with great caution, since vibration could trigger the very landslide that it is intended to prevent. Compaction may be achieved by different techniques, depending on the depth of treatment: compaction with conventional rollers is only effective to less than 1 m and is therefore generally inapplicable to in-situ treatment; depths of 2 – 3m can be achieved by special polygonal rollers, while dynamic compaction carried out by

commercially available equipment can achieve depths of approximately 10 m. For greater depths, in-situ compaction can be achieved by vibrocompaction and associated techniques or by compaction grouting.

- Deep mixing with lime and/or cement is suitable for a wide variety of soils; two basic types of techniques and equipment are available: dry-mix methods use compressed air to deliver the binder to the soil and is most suitable in soft clays with a high water content, such as is encountered for example in Scandinavia and parts of the Far East; wet-mix methods use water to deliver the binder and generally use heavier, more powerful equipment which is better suited less sensitive soils. Traditionally, deep mixing has been carried out with equipment which formed columnar elemnts; in landslide stabilization, it is common to perform compenetrating columns to form panels aligned with the direction of movement. Recent developments include the use of equipment derived from the hydromill used for diaphragm wall construction, allowing direct construction of isolated or compenetrating rectangular panels.
- Permeation grouting, by injecting a low viscosity cementitious or chemical binder in relatively permeable material where it can permeate through the pores without altering the solid skeleton (in soils) or into the discontinuities (in rocks). This minimizes the hazard of triggering the landslide during treatment. The choice of binder depends on the permeability of the medium, but care should be taken in selecting chemical binders to ensure environmental compatibility.
- Compaction grouting, by injecting a high viscosity cementitious binder at high pressure through the tip of the drilling string or, preferably, through pipes equipped with valves (tubes a manchettes) injected one at the time by a system of packers. The expanding grout mass compacts the surrounding granular soils.
- Jet grouting; cementitious low viscosity grout ejected as a high pressure high velocity jet from a nozzle close to the end of the drilling string is used to erode, mix and replace the soil to form a column of soil/grout mix. Excess grout and soil return to the surface along the annulus between the soil and the drilling string. Accidental obstruction of the return path causes pressures in the treatment zone to increase rapidly and must be avoided, using a temporary casing, if necessary. Different technologies exist, using grout only (mono-fluid), grout and air (bi-fluid) and grout, air and water (tri-fluid). Recent advances allow very large (approximately 3 m in favourable conditions) or irregular shaped columns to be formed. The actual column dimensions depend on ground conditions and are not easy to verify. In landslide stabilization, it is common to perform compenetrating columns to form panels aligned with the direction of movement.
- Modification of the groundwater chemistry, by diffusion of lime or salt into the ground. This technique is only suited to treat certain clay slopes and should be considered experimental at this stage.

The common objective of all these techniques is to increase the shear resistance of the ground. However, their applicability to specific cases must be always reviewed with greatr care, since they involve significant associated risks during construction, mainly linked to vibration and the use of heavy equipment, or in the long term, such as, for example, unexpected impacts on groundwater levels.

4.7 TRANSFERRING LOADS TO COMPETENT GROUND

The loads driving instability can be transferred mechanically, in whole or in part, to underlying competent ground by structural elements. Possible techniques include:

- Gravel or concrete filled trenches intersecting the basal failure plane; deep trench drains that intercept the slip plane and provide additional frictional resistance are generally called counterfort drains, although the term is sometimes used loosely to indicate all deep trench drains aligned along or close to the direction of maximum inclination of the slope, irrespective of whether they intersect the slip plane.
- Piles or barretts (diaphragm wall elements), placed either at regular 2D spacing over the whole slide or portion thereof, to act as isolated dowels, or, more commonly, at close spacing along one or more specific alignments to form embedded walls across the direction of movement, in which case they are often supplemented by anchors.
- Large diameter caissons typically ranging in diameter between 6 and 15 m (Brandl, 1988; Leoni and Manassero, 2003). They can be placed in earth and debris slopes, typically along specific alignements across the direction of movement at strategic positions within the landslide, at a maximum centre to centre spacing of twice the diameter. The method of construction depends on ground and groundwater conditions. They can be supplemented by anchors and/or subhorizontal drains drilled form within the caissons themselves.
- Soil nailing, consisting of solid or hollow steel or glass fibre bars grouted into the face of an excavation or an existing slope to reinforce it. The face of the slope is protected by shotcrete and welded wire mesh, geogrid/geotextiles sheets and cast-in-place concrete or prefabricated panels, depending on slope angle and ground conditions.
- Dowels, consisting of short untensioned steel bars inserted and grouted into holes drilled across the potentially unstable block or slab down to the underlaying stable rock; where the mass to be supported is fractured into blocks which are too small to be dowelled individually and/or rests on material which is not sufficiently competent to provide adequate anchorage to the dowels, the potentially unstable mass may be harnessed by structural netting (or, more rarely, ropes) of adequate stiffness and resistance, anchored by dowels along the edges of the potentially unstable mass.
- Rock bolting, consisting of the systematic reinforcment and/or anchorage of rock slopes by the insertion and grouting of un-tensioned (passive) or tensioned (active) steel bars into holes predrilled typically up to 12 to 15 m into the more or less fractured rock mass, improving its stability. Long bolts are typically formed by joining shorter threaded bars using special couplers, to facilitate handling.
- Strand anchors installed and grouted in predrilled holes in soil or rock to transmit an applied tensile load into the ground. They are typically manufactured with high strength low relaxations class 1860 MPa steel in strands 15.7 mm (0.6”) in diameter; the number of strands typically varies from 3 to 8. The maximum length is nominally unlimited, since the strand can be manufactured and assembled in any length and it can be transported coiled; in practice, however, the maximum length is limited by drilling. Typical overall lengths are up to 35 – 40 m.

Mitigation measures in this cathegory operate as a surrogate increase in the resistance of the actual or potential sliding mass either by partially replacing the shear surface with more competent materials (e.g. shear keys, piles, etc. – **Figure 6**) or by mechanically increasing the effective normal stress on the actual or potential failure surface, thus increasing the shear resistance of the soil or rock (eg. pretensione strand anchors – **Figure 7**). Some systems operate on both principles simultaneously (eg. passive anchors, soil nailing, rock bolting – **Figure 8**). In all cases, these measures operate by transferring part of the driving forces to the more competent, stable strata underlying the (actual or potential) sliding mass.

These systems progressively loose their effectiveness as the sliding mass becomes a flowing mass, either through internal processes (eg. loss of microstructure, especially in saturated materials), or through mixing with addition of water from surface runoff or graoundwater.

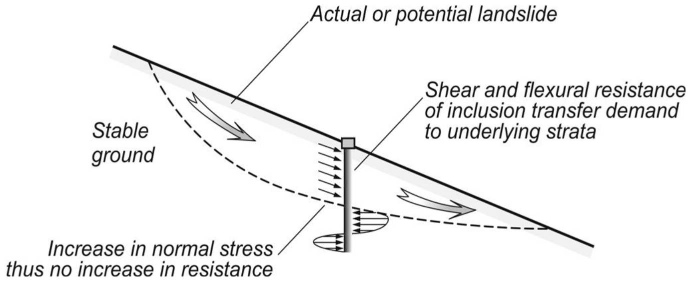


Figure 6: Load transfer by systems acting in shear and bending

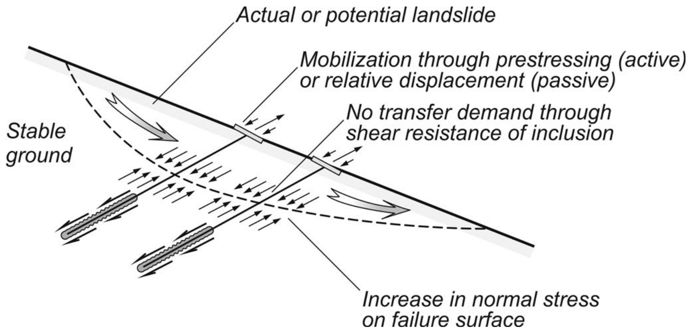


Figure 7: Load transfer by systems acting in tension to increase the effective normal stress on the failure surface

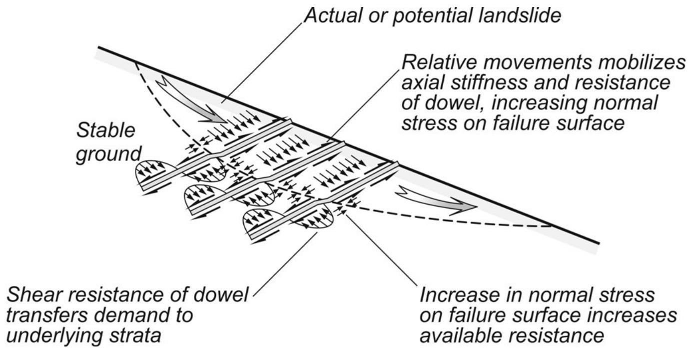


Figure 8: Load transfer by mixed systems acting in shear, bending and tension

4.8 RETAINING STRUCTURES

Retaining structures are used extensively and can be considered as an additional class of hazard mitigation measures to prevent landslide triggering, even though they are used as means to modify slope geometry and/or to transfer load to more competent strata, rather than to address a specific physical process.

Retaining structures may provide a workable solution where conventional filling at the toe of the slope is not feasible due to geometrical constraints or due to interference with existing structures or infrastructure; depending on their configuration and their location in relation to the landslide mass, they permit construction of toe weighting with a reduced landtake and/or to transmit horizontal forces to competent foundation material in front of the toe.

Retaining walls may be substantially of three types (**Figure 9**):

- Cantilever walls;
- Gravity walls, including masonry, mass concrete, crib walls, gabion walls and similar;
- Reinforced soil systems.

As a general rule for slope stabilization, relatively flexible retaining structures should be preferred to rigid structures, which are less tolerant to differential displacements.

Systems such as crib walls, gabion walls and the various types of reinforced soil systems are increasingly common.

Similar structures can also be used as protective barriers, to intercept or redirect the run-out of rockfall, flow-slides and avalanches.

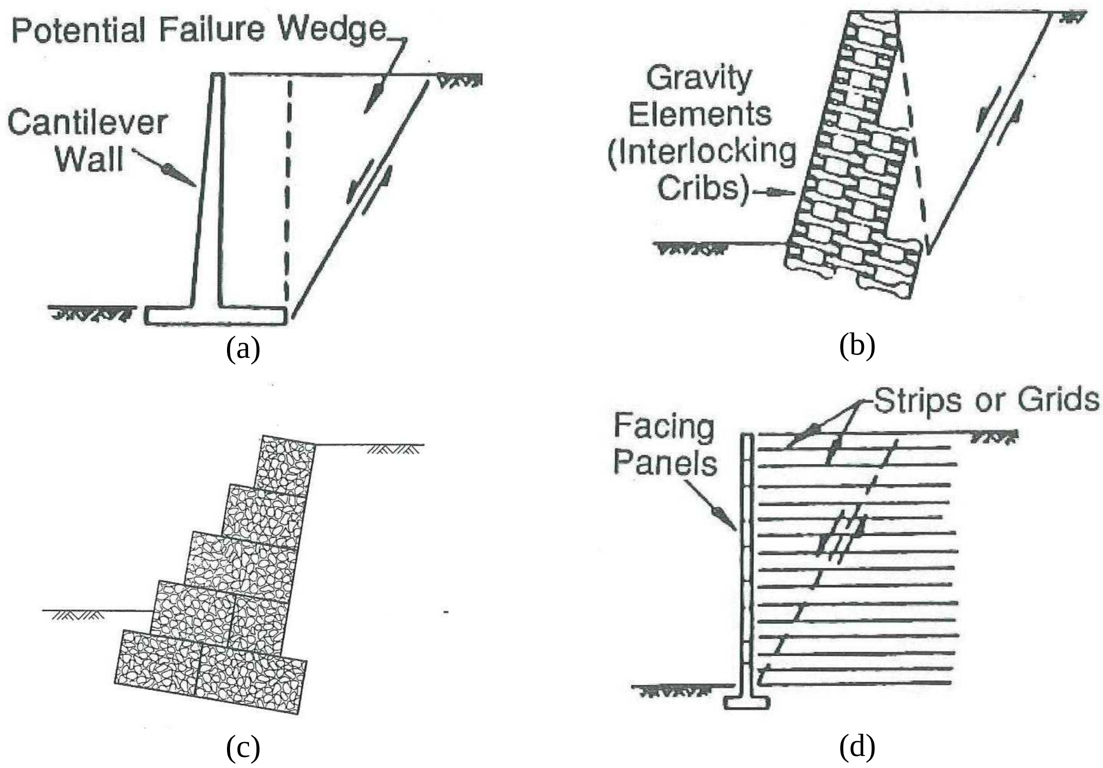


Figure 9: Typical retaining structures: a) gravity walls; b) crib walls; c) gabion walls; d) reinforced soil walls (modified after Holtz and Schuster, 1996; GEO, 1993)

4.9 REVIEW OF MEASURES INVESTIGATED WITHIN PHYSICAL MODELS, WITH RECOMMENDATIONS FOR DESIGN

4.9.1 Interaction of a pile row with an unstable soil lay (based on Yoon & Ellis, 2010)

Yoon & Ellis (2010) performed a series of centrifuge model tests to study the interaction of a pile row with an unstable soil layer. The spacing between the piles is typically 3–4 diameters in the row. The piles act in shear and bending to resist the passive lateral load applied from the unstable part of the slope (**Figure 10**(a)). The larger the spacings between the piles, the more economical this approach is. But there is an increased risk of ‘flow’ of the unstable soil between the piles, as shown in **Figure 10**(b).

The relative soil- pile displacements were measured using the Particle Image Velocimetry technique (**Figure 11**). Occurrence of bulging at the toe of the slope, and just upslope of the pile row, indicates impedance of the pile row against downslope soil movement.

A variable B_{mob} was used to express the equivalent lateral pressure mobilised on the pile (p = load per unit length/diameter) due to interaction, relative to the nominal overburden stress at a given depth in the unstable soil layer ($\sigma'_v = \gamma z$).

$$B_{mob}(z) = \frac{p(z)}{\sigma'_v(z)} \qquad (1)$$

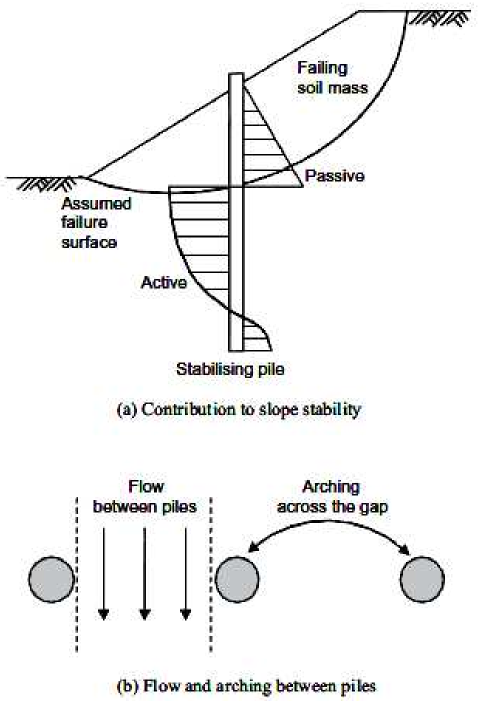


Figure 10: Slope stabilisation using a discrete bored pile wall (Yoon & Ellis, 2010)

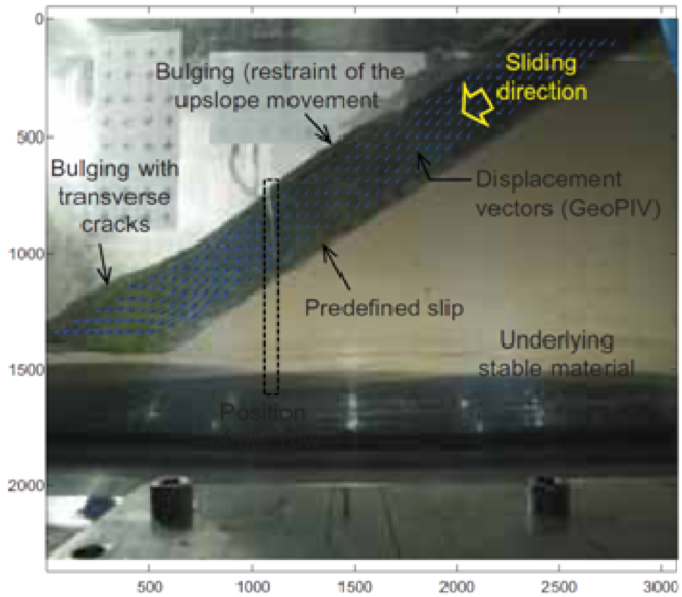


Figure 11: Side view of test with stabilising discrete pile row at 50 g (Yoon & Ellis, 2010)

The mobilised and normalised interaction pressure B_{mob} was found to be approximately constant with depth and this was also the case throughout the centrifuge test (as the g-level increased). Except at pile spacings less than a critical value (3 times the pile diameter) proposed by Durrani et al. (2008), B_{mob} tended to a maximum value of approximately K_p^2, corresponding to an isolated pile. The maximum value of B_{mob} does not appear to be significantly increased due to the inclination of the upslope soil loading the piles, and it is conservative in any case to ignore this potential effect in design. The results have verified that the K_p^2 limit on interaction for an isolated pile can be used to propose the critical pile spacing, where arching is effective.

4.9.2 Full-scale reinforced soil retaining wall under dynamic loading (based on Ling et al., 2003a,b & Mayne et al., 2009)

Ling et al. (2003a) described the behaviour of a full-scale reinforced soil retaining wall, subjected to earthquake shaking, for validation of numerical analyses. The wall was instrumented with transducers and was 2.8 m high (**Figure 12**), which is the deepest soil model reported to have undergone excitation on a 1g shaking table to date. Kobe earthquake motions were simulated to excite the wall at a maximum base acceleration of 0.4g initially, followed by 0.8g. The wall withstood the initial shaking (0.4g) with minimal deformation and an acceleration amplification of 1.35. The wall deformations, settlements and acceleration amplification were almost negligible (**Figure 13**).

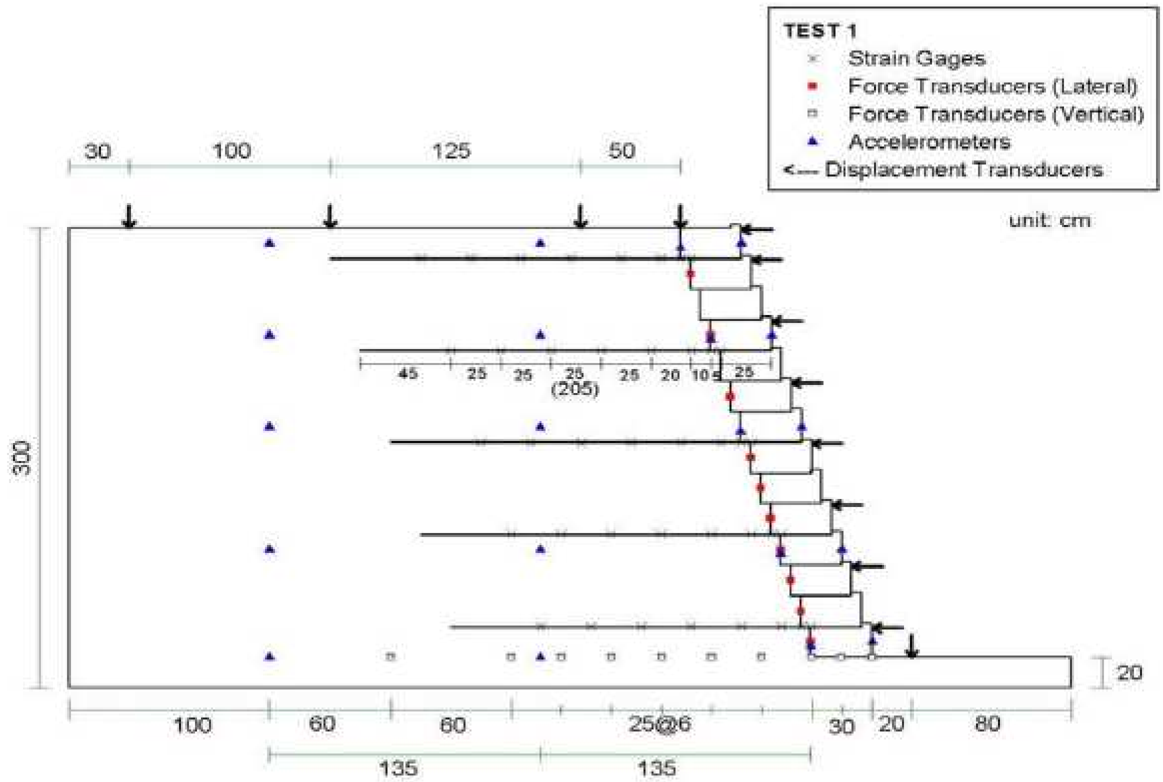


Figure 12: Cross section through an instrumented geogrid reinforced modular block retaining wall (Ling et al., 2003a)

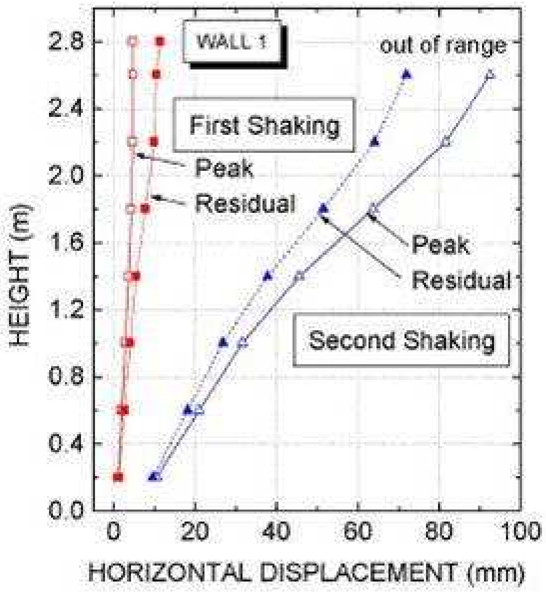


Figure 13: Horizontal displacement of the front of the retaining wall with height for both phases of shaking (Ling et al., 2003a)

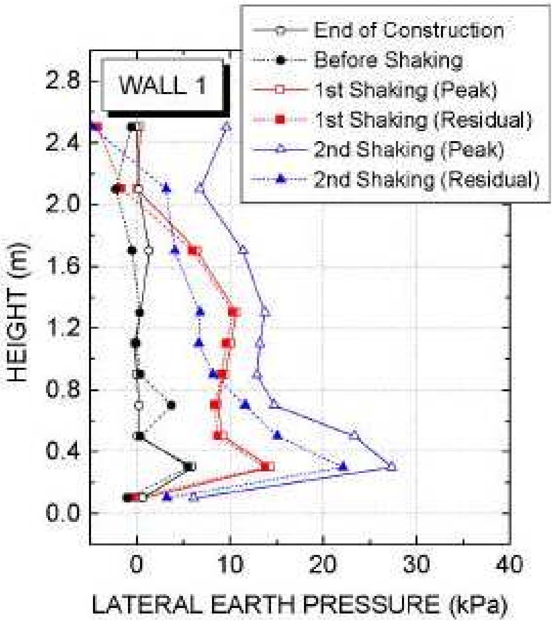


Figure 14: Peak and residual lateral earth pressure acting over height of retaining wall for the shaking phases (Ling et al., 2003a).

Slightly larger horizontal deformations and settlements were observed during shaking with a peak acceleration of 0.8g, and the tension mobilised in the bottom two reinforcement layers increased noticeably. Lateral earth pressure (**Figure 14**) acting behind the wall was only marginally larger at these lower depths, and the wall remained stable and serviceable. The 1g shaking table tests confirmed that the modular block system interacted effectively with the geogrid reinforcement to render this wall system stable when subjected to significant earthquake loading.

Leshchinsky et al. (1995) presented a unified design approach, based on limit equilibrium analysis that considers the various aspects of stability of reinforced soil structures including the stabilizing effects of facing blocks. The design procedure is validated using the test results of full-scale walls. They concluded that the facing interblock friction significantly reduces the required geosynthetic length and strength for a near-vertical wall. This effect diminishes as the slope angle reduces.

Ling et al. (2003 b) suggested that the wall facing contributes to a better performance, in terms of deformation and acceleration response. Rigid facings were found to perform better than the discrete wall panel.

4.9.3 Centrifuge modelling of reinforeced soil structures (based on Zornberg et al., 1998a&b, Mayne et al., 2009, and Springman et al., 1997)

A centrifuge study was conducted by Zornberg et al. (1998a&b) to evaluate the suitability of current design methods of reinforced soil structures. The results of this investigation indicated that the orientation of reinforcement forces should be considered to be horizontal, that significant contribution to stability is provided by the overlapping reinforcement layers, and that rigorous limit equilibrium analyses can predict the collapse of reinforced soil structures accurately when using the soil peak shear strength in the analysis (Zornberg and Arriaga, 2003). The location of the failure surface observed experimentally was accurately predicted by limit equilibrium approaches currently used in design (**Figure 15**). These findings support earlier findings by Springman et al. (1997), who instrumented geosynthetics with strain gauges and investigated deformation mechanisms arising during increase of gravity and vertical loading on top of the wall. Subsequent centrifuge studies conducted by Viswanadham and Mahajan (2007) confirm, using digital image analysis, the suitability of current design methods for geosynthetic-reinforced soil structures (Mayne et al., 2009).

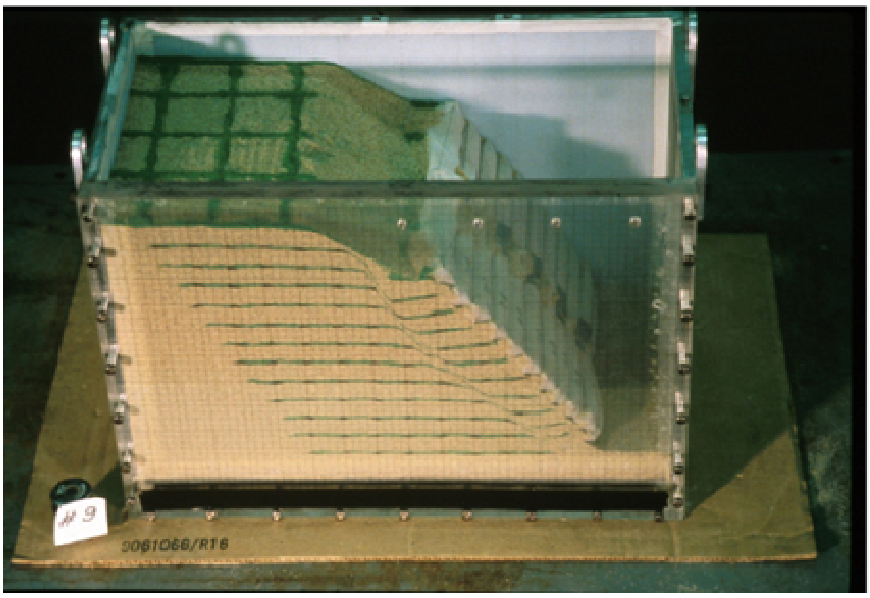


Figure 15: View of failure surface in the reduced-scale model of a geosynthetic-reinforced soil structure after testing in a geotechnical centrifuge. (Zornberg et al., 1998).

4.9.4 Rainfall induced landslides (based on Take et al., 2004)

Take et al. (2004) performed a series of physical model tests, to evaluate two candidate triggering mechanisms of fast landslides in decomposed granite fill slopes against observations of slope behaviour in centrifuge model tests.

Despite observing significant collapse due to wetting in an unsaturated loose fill (**Figure 16**), excess pore pressures were dissipated in the voids of loose structure of the soil. More danger was witnessed for constricted flow in layered slope systems, which resulted in transmission of a slow moving slip into a rapid flow through localised transient pore pressure rise even in densely compacted fills (**Figure 17**). It was concluded that the priority in hazard reduction of loosely compacted fills should be in preventing the build up of localised pore pressures through permeable layers. Interception of groundwater percolation would be more useful than densification as a remedial measure, although the removal, mixing, and compaction of loose fill would have the coincidental benefit of eliminating permeable layers. Attention should be focussed particularly on regions of slopes where springs of seepage are observed after rainstorms. Shallow horizontal drains should be particularly effective in suppressing slip triggering in such locations

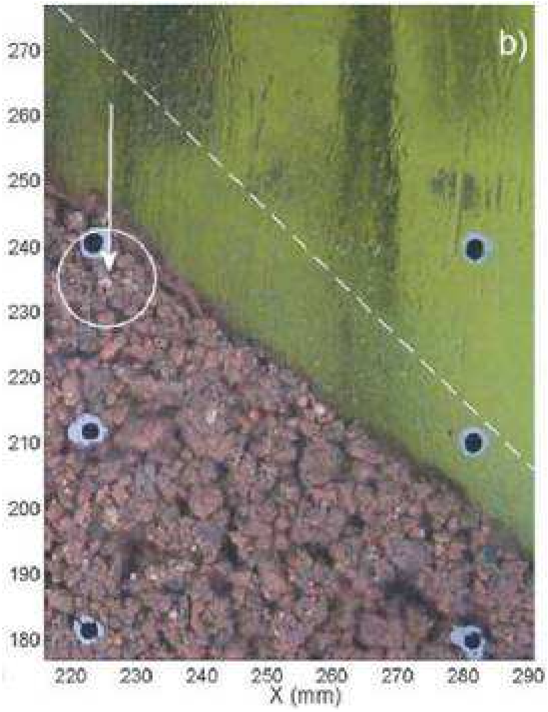


Figure 16: Moist tamped loose fill after rainfall in a beam centrifuge (Take et al., 2004).

Figure 17: Transition from slide to flow in dense fill model (Take et al., 2004)

4.9.5 Thawing of ice in rock joints (based on Günzel & Davies, 2006)

Günzel & Davies, (2006) performed a series of centrifuge tests on instrumented rock slopes with an ice filled joint. The rocks were reinforced by pre-stressed rock bolts. The ice was allowed to thaw during the test.The stress development and settlement was monitored during the experiment with thermocouples, load cells and LVDTs (**Figure 18**, **Figure 19**).

They concluded that warming of ice inside a joint could lead to a significant drop of factor of safety compared to a joint filled with cold ice or without any ice. However, they observed that using pre-stressed rock bolts can be a good approach to stabilise these discontinuities. These bolts will have to be tested regularly, as they might lose tension as the joint closes.

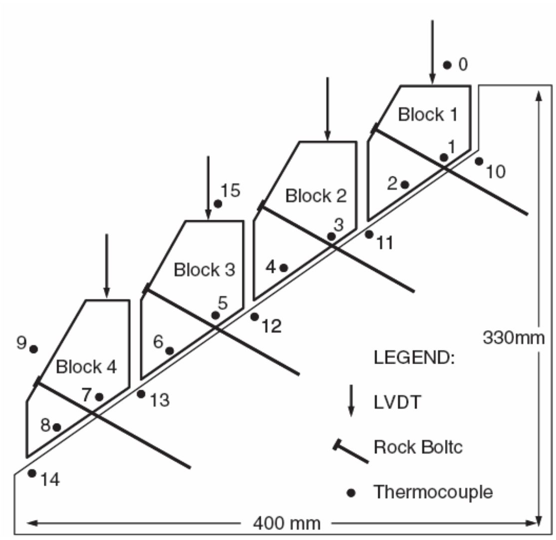


Figure 18: Loose, faulted rock anchored by rock bolts through ice filled joint (Günzel & Davies, 2006).

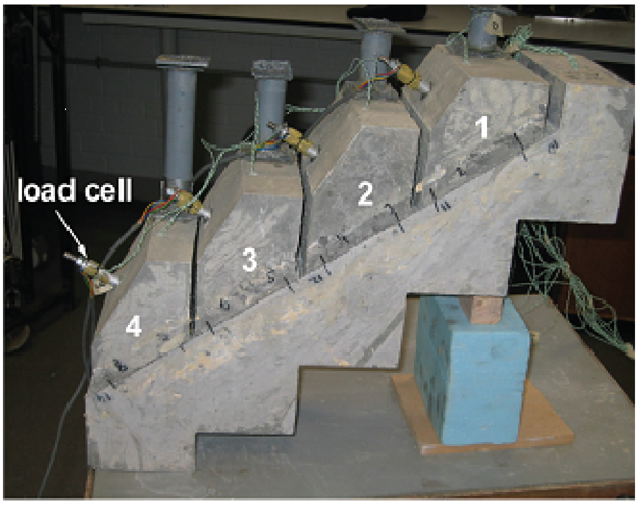


Figure 19: Model slope; four separate blocks above joint with saw tooth surface (Günzel & Davies, 2006).

4.9.6 Stabilisation effects of plant roots

Sonnenberg et al. (2010) performed a series of centrifuge model tests to study the stabilisation effects of plant roots in (45°) compacted clay embankments (**Figure 20**, **Figure 21**). The embankments were brought to failure by increasing the height of the internal water table. The authors compared the collapse behaviour of unreinforced slopes to that of those reinforced by root analogues or real willow roots (**Figure 22**). The change in FoS could be estimated by comparing the calculated factor of safety (FoS) for the reinforced tests with those from the fallow (control) test. Thus, an improvement ratio (i.e. the difference in FoS between the fallow and reinforced test divided by the fallow test FoS) was defined to quantify this reinforcement. The estimated improvement ratio from tests with root analogues was found to be in the range of 5% to 25%, but was lower in the tests with grown willow roots. The experimental methodology should be used to investigate further the interaction of roots with soil.

Figure 20: Slope model with willows grown for 290 days (Sonnenberg et al., 2010).

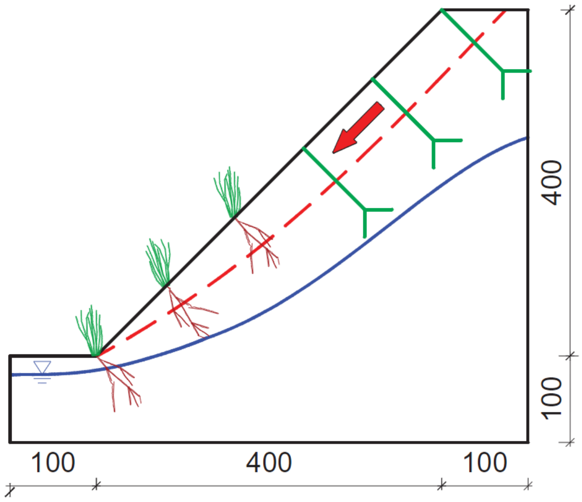
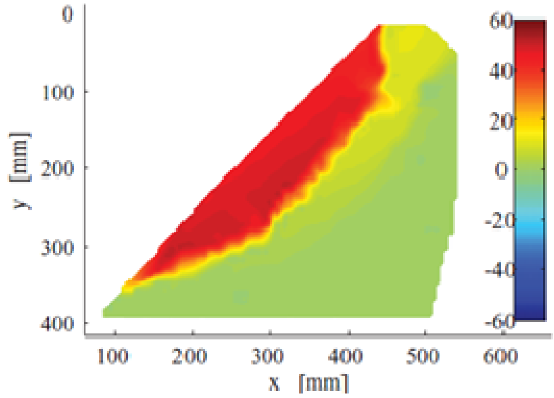
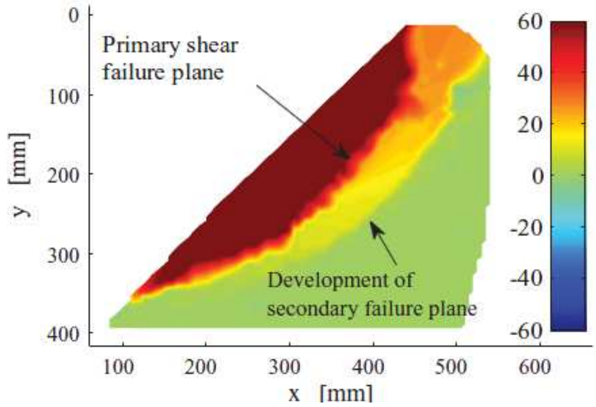


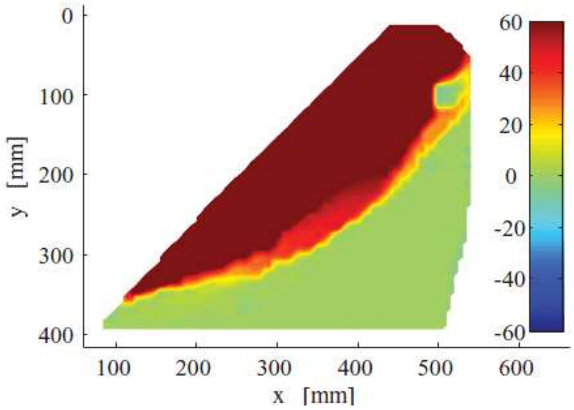
Figure 21: Schematic of the slope model with root analogues and real roots (dimensions in mm) (Sonnenberg et al., 2010)



(a) Shallow slope failure development across roots



(b) Beginning of shear plane migration.



(c) Final shear plane below root influence

Figure 22: Contours of displacement magnitude (mm in model scale) at different stages of test Wooden taproot (Sonnenberg et al., 2010).

4.9.7 A method to monitor the integrity of ground anchorages (based on Hao et al., 2006 & Palop et al., 2010)

Current guidelines for ground anchorages indicate that their integrity should be monitored by using load cells or hydraulic jacks (BSI, 1989), resulting in only a small percentage of the anchorages installed in practice being monitored, because these techniques are either expensive or may lead to damage of the anchorage. A new system, GRANIT (GRound ANchorage Integrity Testing), has been developed based on observing the dynamic response from anchorages, to which an impulse of known intensity has been applied.

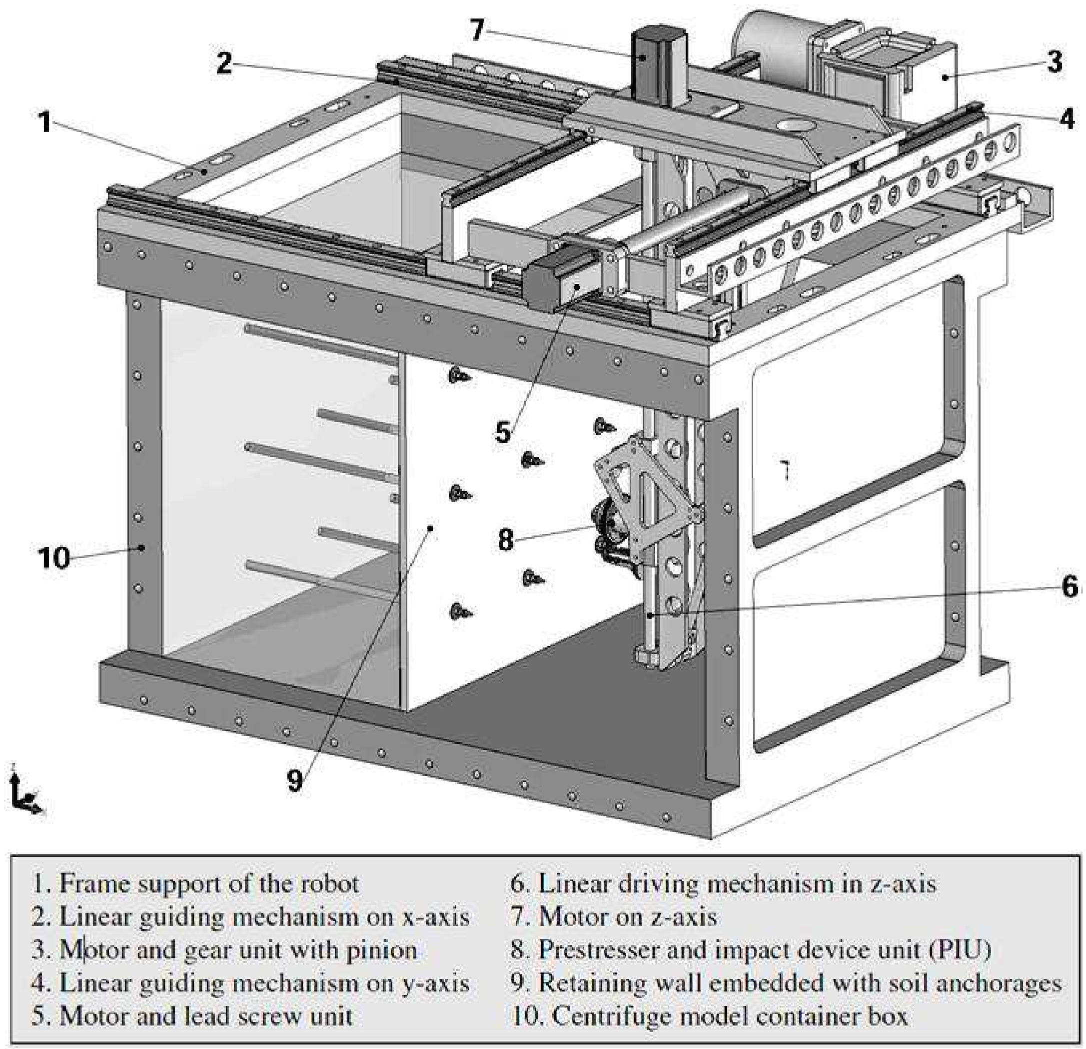


Figure 23: Schematic diagram of the centrifuge testing system (Hao et al., 2006)

Hao et al. (2006) demonstrated a centrifuge testing system to conduct such tests (**Figure 23**). A purpose-built testing robot applies an impulse loading to the head of any anchorage on the retaining wall, and then the strain gauges and accelerometer capture dynamic responses of the anchorage system, where the anchorage can be tightened to different pre-stress levels, as required, in-flight.

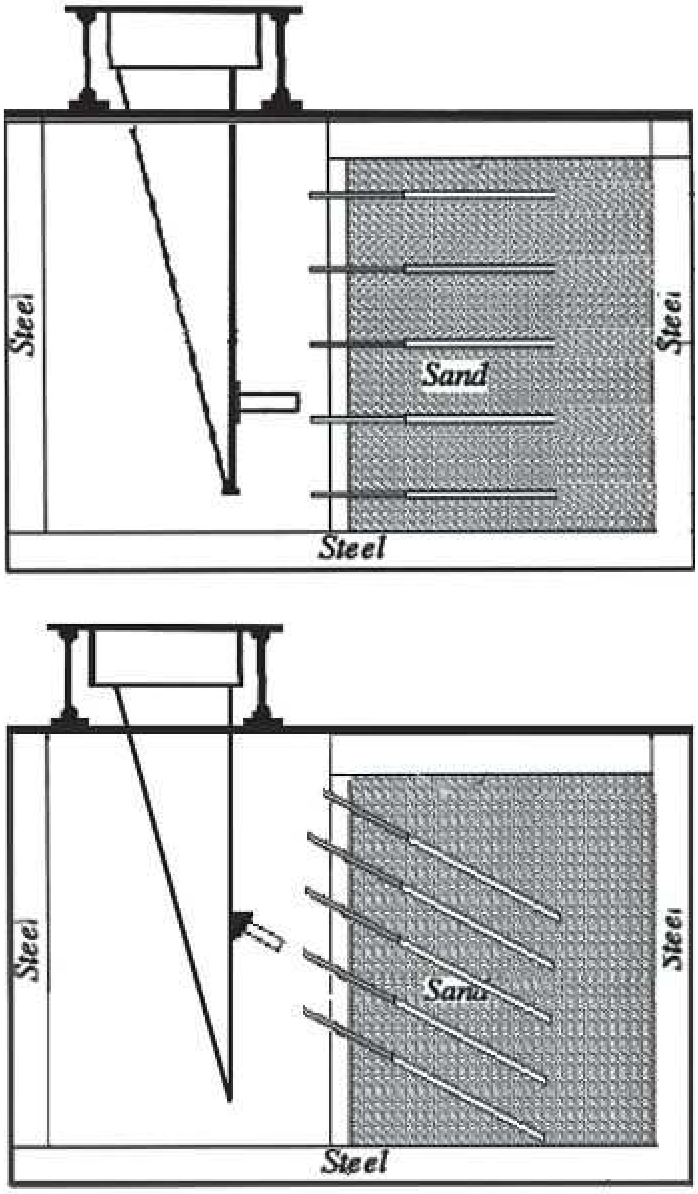


Figure 24: Configuration used for testing horizontal soil anchorages in the centrifuge (above) and inclined anchorages (below) (Palop et al., 2010)

The results and analysis from the two different test configurations (**Figure 24**) presented by Palop et al., (2010) indicate that similarities between rock and soil anchorages can be drawn in terms of potential detection of load although the changes in frequency, related to the soil anchorages, are much smaller than those related to the rock anchorages. This has implications for the use of dynamic testing for soil anchorages where it may be necessary to tune the impulse to optimize the dynamic response. This may be achieved either by an increase of the load applied or modification of the frequency content of the impulse applied. The results from these scaled tests can be used to interpret better the results of tests on full scale soil anchorages.

4.9.8 Soil nailing (based on Davies & Jones, 1998 & Junaideen et al., 2004)

Davis and Jones (1998) performed a series of centrifuge tests to investigate the effects of nail orientation and contribution of the stiffness of nails to the stability of slopes. The prototype of these tests was a 70° and 3 metre high cutting supported by three rows of 40 mm diameter nails (**Figure 25**). In most design codes of soil nailing systems, the process of construction (i.e. loading path followed during construction) is not considered. They simulated the excavation process by draining a solution of zinc-chloride. The results of their tests showed that the stiffness of the nails did not appear to have a major effect on the overall stability of the slope, which is provided under working conditions by axial load transfer in the nails. The facing (even flexible ones) assisted the load transfer from active zone to the resistant zone via the nails.

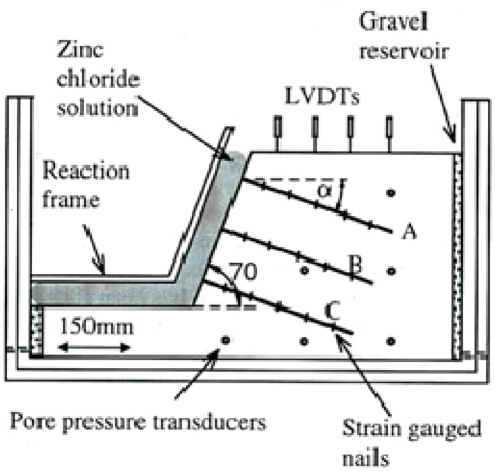


Figure 25: Centrifuge model layout (Davies & Jones, 1998)

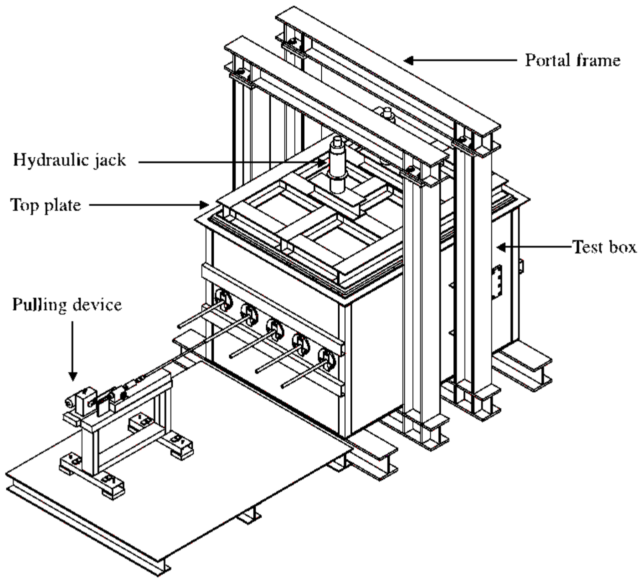


Figure 26: Pulling test apparatus (Junaideen et al., 2004)

Junaideen et al. (2004) built a large-scale laboratory apparatus (2 m long, 1.6 m wide, 1. 4 m high, **Figure 26**) to study the soil–nail interaction in loose fill materials. Pullout tests were performed with a contolled displacement-rate on on three types of steel bars (ribbed bars, knurled bars, and round smooth bars) embedded in loose, completely decomposed granitic soils. The results showed that the normal stress acting on the nail increases (decreases) due to the dilative (contractive) tendency of the soil being sheared in the pre-peak states and decreases due to the arching effect of the soil in the post-peak states. The ribs have a significant influence on the pullout resistance. The results of pullout tests carried out in a multistage manner show that the increase in pullout resistance of the ribbed bars is not significant with an increase in the applied overburden pressures. The conventional method of analysis tends to give a low interface friction angle and high interface adhesion. The correct interface parameters can be determined by taking into account the changes in the normal stress acting on the nail.

5 MEASURES TO REDUCE VULNERABILITY

5.1 GENERAL

Measures to reduce the vulnerability of the elements at risk consist of “passive” solutions which are not intended to prevent the triggering of the landslide but to reduce the resulting degree of loss. They can be subdivided in two main categories, depending on the approach followed to achieve this objective:

- **Measures to increase the resistance of elements at risk** (reduction of vulnerability s.s.) – existing structures can be strengthened; for new structures, the potential effects of impact from landslide material can be taken into account from the outset. This approach is typically applicable only in relation to relatively shallow slides, since it is practically imposible to buid structures capable of withstanding the impact form larger landslides.
- **Measures to stop or to deviate the path of the landslide** debris (reduction of vulnerability s.l.) - Works can be carried out to intercept and block or at least to deviate or to slow down the sliding materials. This type of works relates mainly to the fall of massive blocks or to flows of all types, in those cases where a large slope is affected and stabilization is not feasible for environmental impact reasons or because of cost.

5.2 MEASURES TO IMPROVE THE RESISTANCE OF ELEMENTS AT RISK

Measures to reduce the physical vulnerability of buildings and infrastructures by increasing their resistance are commonly referred as strengthening of existing RC or masonry structures. For new constructions the strengthening is part of their design philosophy and construction practice in order to accommodate with safety, in landslide prone areas, the estimated permanent ground deformations.

The aim is to resist the impact from the sliding or rotating ground (rock or soil) mass minimizing the physical losses and casualties. The basic idea is to design the foundation and the rest of the bearing elements of the structure in such a way that they can withstand the landslide movement (permanent displacements) and/or the landslide and rock fall impact with little or repaired damages.

The first goal is to save lives and then to save the integrity of the structure, which may accommodate a certain level of repairable damages. Once the first goal is achieved then the selection of the strengthening method may be evaluated on a cost benefit basis. Excessive cost may lead to the radical decision to withdraw the strengthening solution and move to a completely new structure in another safer place.

For the cases under consideration the sliding material consists either of falling or tumbling rocks and massive debris flow or rotational and translational slow moving soil slides. Strengthening of structures is generally meaningful in case of rather shallow landslides. It is clear that in cases of very deep massive slides of considerable moving mass, the strengthening approach is of limited applicability. It is also true that in several cases the strengthening approach, which actually consists of increasing the resistance and stiffness of the construction, is of limited practical importance since the cost of the works that would be required may be higher compared to the cost of relocating the structure (excluding the cost of the terrain). Strengthening approaches are interesting in case of moderate landslides, which are developed slowly in time, and of course in cases of structures and urban centres of major importance, for which relocation is not feasible for several reasons including historical and archeological ones. Strengthening of structures is also a good approach in case of moderate size rock falls and equally small to moderate size earth flow.

Another important parameter affecting the strengthening approach is the fact that any approach is strongly case depended, in the sense that the chracteristics of the structure, its relative position within the landslide zone and the soil-rock properties play an important role in any strengthening decision. This is why the relevant research and design practice is rather fragmented. Consequently it is practically impossible to define, in a general way, the improvement in the vulnerability (in quantitative terms) obtained by increasing the resistance of different parts of the structure, because de facto this is a case depended evaluation.

Among the few methods available to evaluate the response of reinforced concrete buildings in case of rockfall is that recently proposed by Mavrouli and Corominas (2010). The methodology can be applied to evaluate the necessary strengthening of a specific structure affected by rock falls. Although the procedure is rather limited for the moment to specific cases, (i.e. 2 storey RC buildings), it is certainly promising. Similar methods have been proposed in the case of rockfall impact on road and railway infrastructures. Methods used to estimate the impact of avalanches and/or lava flows on buildings could be also applied in cases of massive fast land movement. A rather comprehensive description of these methods may be found in Pudasaini and Hutter (2007). In the framework of the present research project a comprehensive methodology has been proposed by Fotopoulou and Pitilakis (2011) to assess the vulnerability of simple RC structures to relative slow moving earth slides, as described in SAFELAND Deliverable D2.5 on “Physical vulnerability of elements at risk to landslides: Methodology for evaluation, fragility curves and damage states for buildings and lifelines”. The method proposes fragility curves for two types of foundation systems, flexible and stiff. It is thus possible to evaluate the benefit that may be obtained from a strengthening applied to the foundation system by comparing, for a given seismic intensity, the fragility curves of the two different foundation systems. A similar approach may be applied for other triggering mechanisms.

In the ensuing subsection, mitigation measures to reduce vulnerability through the increase of the resistance of the elements at risk (buildings and infrastructures) for slow moving earth slides are presented.

For a shallow, relative slow moving landslide, the strengthening of the exposed structure should be design in order to decrease its vulnerability. By upgrading the geometrical and material properties of the exposed building, the quality of maintenance, the code design level, certainly the local soil and drainage conditions, as well as the foundation and structure details, it is feasible to increase its resistance of the structure to withstand the estimated amount landslide permanent displacement with limited damages.

The capacity of the structure to resist the permanent ground deformation depends primarily on the foundation type. A structure with a deep foundation (e.g. piles) compared to shallow foundations often experiences higher resistance ability and hence a lower vulnerability. For shallow foundations, the distinction is between rigid or flexible/unrestrained foundation systems.

When the foundation system is rigid (e.g. continuous mat foundation), the building is expected rather to rotate as a rigid body and a failure mainly attributed to the loss of functionality of the structure is anticipated. On the contrary, when the foundation system is flexible (e.g. isolated footings), the various modes of differential deformation produce structural damage (e.g. cracks) to the building members (**Figure 27**).

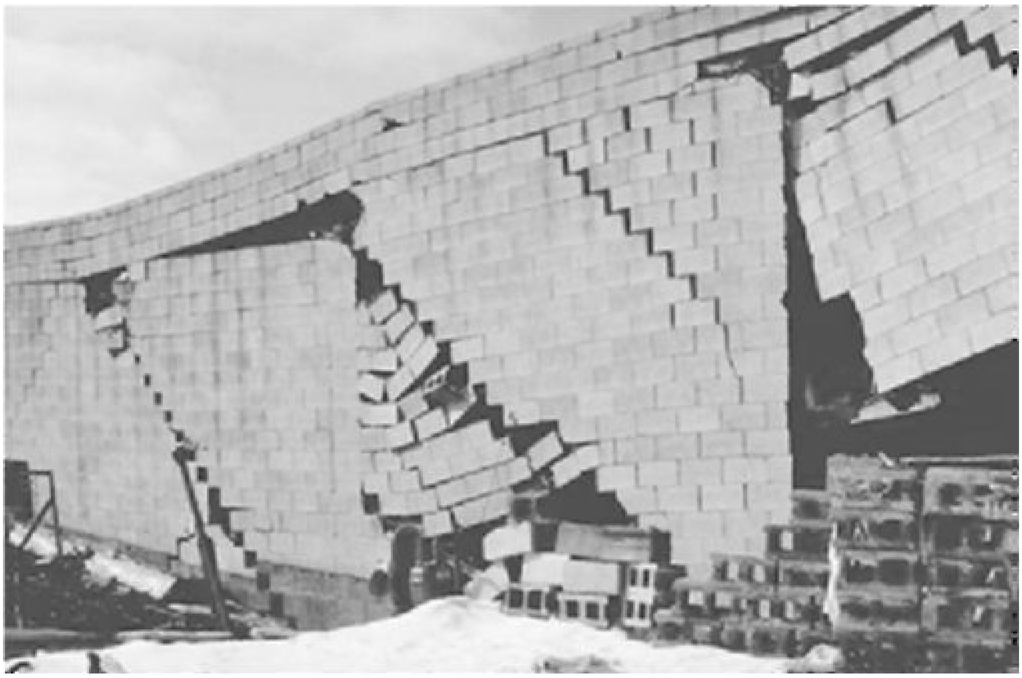


Figure 27: Typical Building damage caused by the Fourth Avenue landslide, Anchorage, Alaska. The landslide movement occurred during the Prince William Sound earthquake in Alaska on March 27, 1964. (Photograph from the Steinbrugge Collection, EERC, University of California, Berkeley) (Day, 2002)

In order to apply any mitigation measure that will result in the reduction of vulnerability of the affected buildings and facilities, first of all, the landslide displacement potential should be adequately predicted. Accurate estimating of the ground displacements evaluated with time, requires sufficient geological, geotechnical surveys, field measurements and adequate laboratory testing.

Measures to reduce the vulnerability through the increase of structural resistance may be summarized as follows:

- Strengthening of shallow foundations and improved structural design to withstand predicted permanent ground displacements;
- Deep foundations properly designed to accommodate the landslide effect;
- Deep anchoring of foundation elements;
- Combination of the above three approaches.

The typical grading solution to this type of failure is first to estimate the amount of potential landslide displacement, and then to design and construct a mat of compacted fill that is thick enough to form a uniform bearing surface. Designing the thickness of the mat foundation is intended to accommodate different amounts of displacements, mainly differential. The best technique is to remove the surface soil to a certain depth in order to find a better foundation soil and a more stable subsoil conditions. However an often-used practice is to actually construct the mat foundation on the existing ground level after a minimum leveling and compaction (**Figure 28**) instead of excavating below grade. In general, the thicker and stiffer the mat, the greater amount of displacement it can accommodate. The depth of the foundation mat depends on the water table and in general a raised mat has the added impact of providing greater separation from a shallow water table (CGS, 2008).

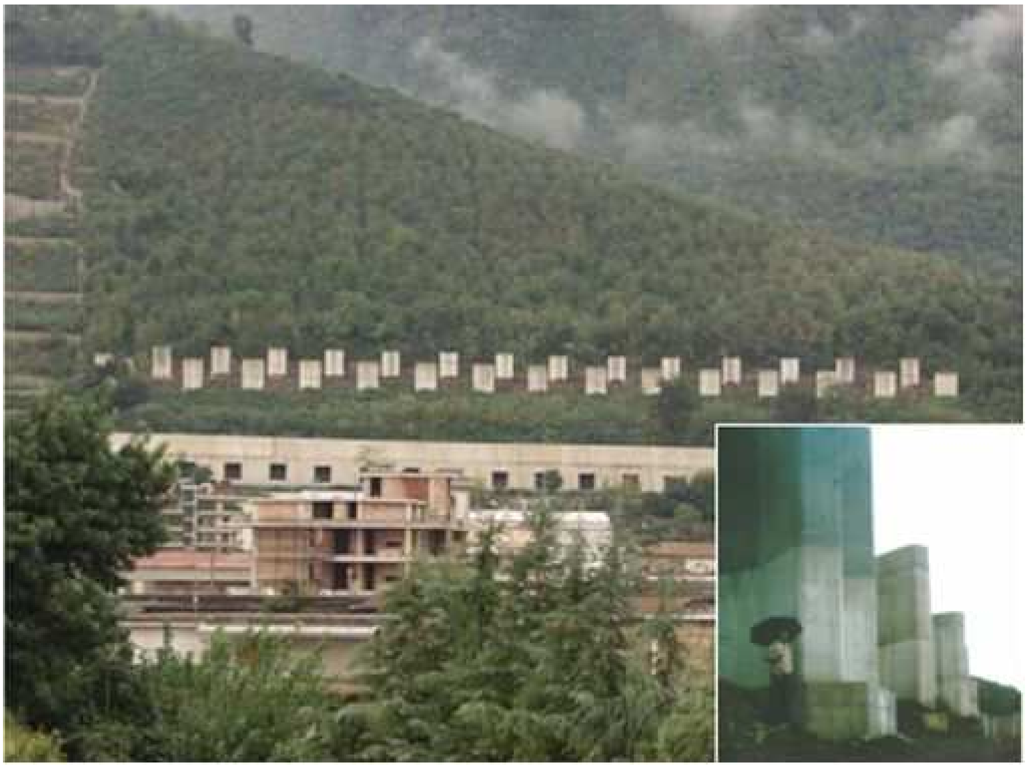
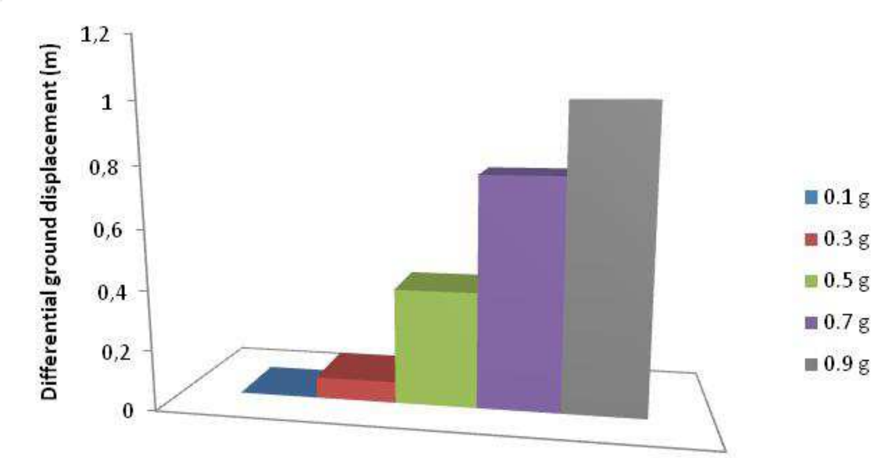


Figure 28: Illustration of a constructed raised mat foundation in Italy.

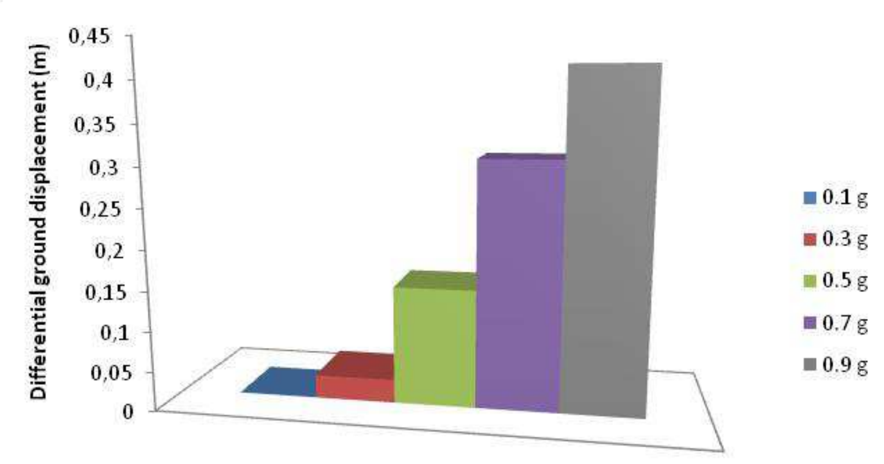
To illustrate the positive effects of strengthening in reducing the vulnerability, the next paragraph presents the difference in the vulnerability of a single story RC frame building when a flexible foundation (isolated footings) is strengthened and transformed in a stiff mat foundation. The corresponding building is assumed to be located near the crest of a relative slow moving, earth slide. An earthquake triggering mechanism is considered.

Figure 29 illustrates the reduced differential displacement potential for the reinforced building associated with the continuous mat foundation in comparison with the initial building with the flexible foundation system (isolated footings), for different levels of earthquake demand (in terms of PGA).

Figure 30(a) presents the improved structural response (in terms of maximum steel strain) for the building with stiff mat foundation, leading to a considerable reduction in the building's vulnerability, as shown in **Figure 30**(b). In particular, it is observed that the building with the strengthened shallow foundation is anticipated to sustain only minor and moderate structural damage while all damage levels are possible for the initial flexible structure.



(a)



(b)

Figure 29: Maximum values of differential displacement vector at the foundation level for (a) buildings with initial flexible and (b) strengthened, stiff foundation system

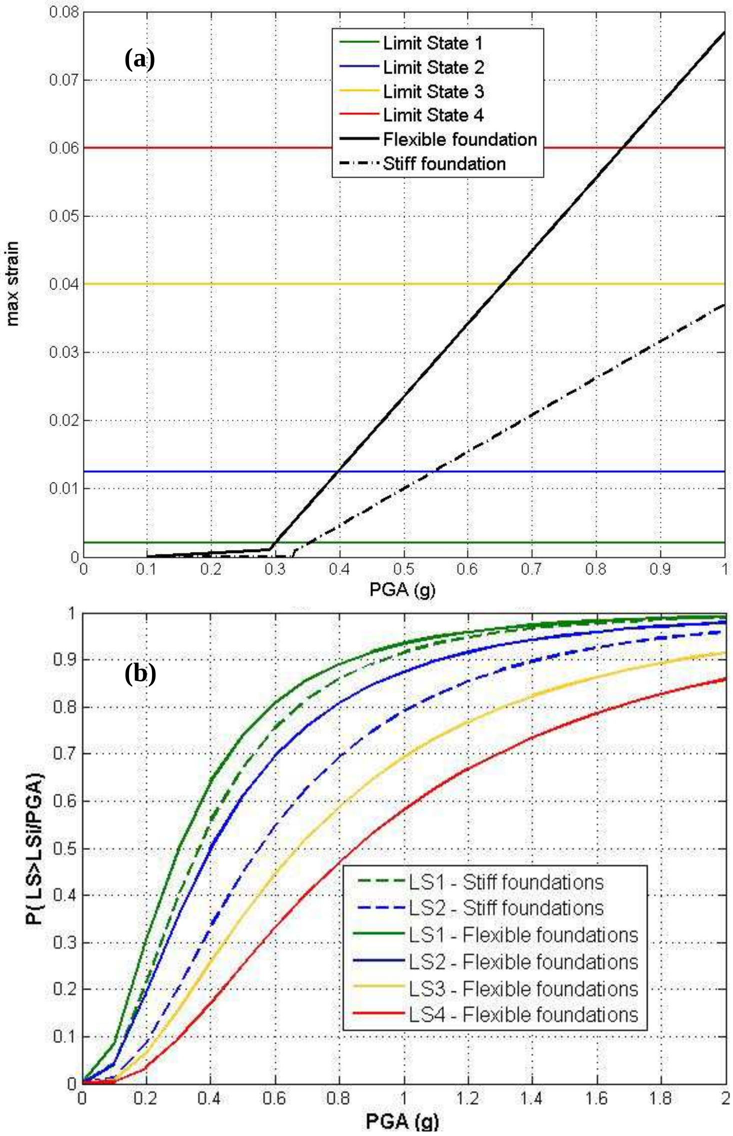


Figure 30: (a) PGA–damage index relationships for low rise RC frame buildings with initial flexible and strengthened, stiff foundation system and (b) corresponding fragility curves.

Anchoring foundations in deeper, more stable soil layers by using piles or caissons can also increase landslide protection. Such designs should take into account the possible down drag forces on the foundation elements due to deformation within the landslide upper soils. A more detailed description of the aforementioned design methods is provided in Section 4.7.

It should be recognized that structural mitigation might not reduce the potential of the soils to slide. There will remain some risk that the structure could still suffer damage and may not be useable if a landslide occurs. Repair and remedial work should be anticipated after a landslide event if mitigation through reduction of vulnerability is used. An illustrational example is provided in **Figure 31**.

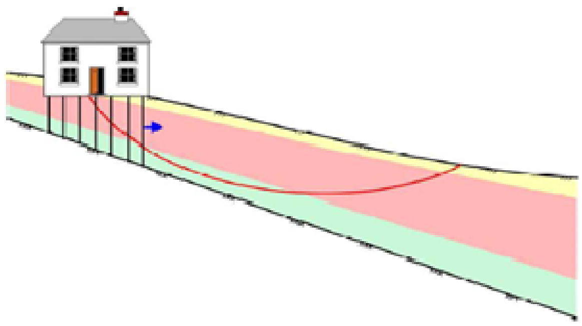


Figure 31: Mitigation through reduction of vulnerability in Drammen (Lacasse et al, 2010) . Sketch of building designed on a deep foundation.

For new structures an adequate setback from the potential precarious slope should be ensured. Uniform Building Code (ICBO, 1997) provides guidance for the general geometry for setbacks (**Figure 32**). In any case, considerable engineering and geologic judgment is also required for each site.

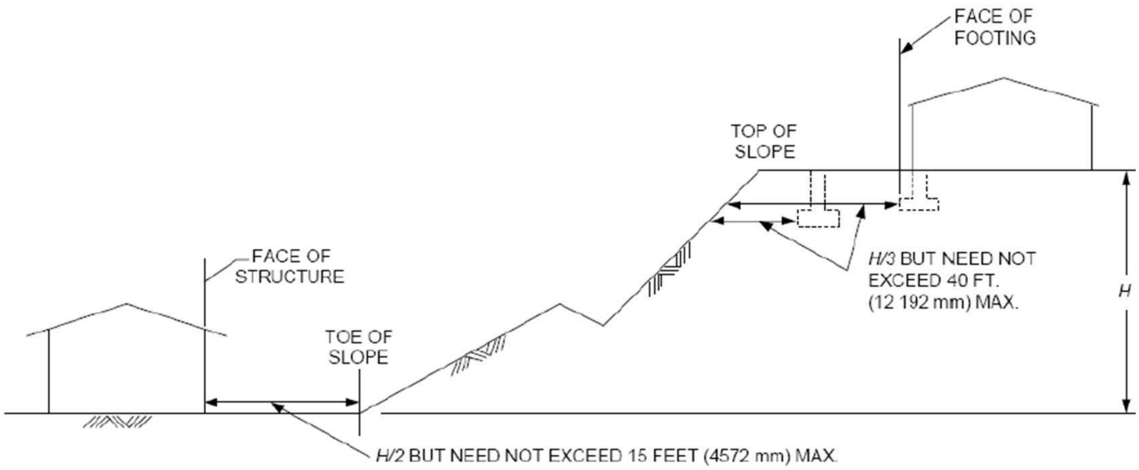


Figure 32: Minimum Setbacks established by the California Building Code (ICBO, 1997)

In conclusion

- For deep-seated slope instability, the strengthening approach by increasing the stiffness of the foundation and maybe of the superstructure as well, is generally not by itself an adequate mitigation measure to reduce efficiently vulnerability.
- The strengthening approach is efficient in case of rather shallow and slowly moving landslides, or in case of moderate rock falls and earth/debris flow.
- The design of any strengthening technique is practically case depended.
- When human lives and casualties are not included among the exposed elements at risk strengthening has to be seen in terms of cost-effectiveness..
- When reducing or avoiding casualties is the main issue, strengthening is always an efficient technique to reduce the physical vulnerability of the exposed elements at risk (i.e. buildings), which implicitly reduces the vulnerability of the non-physical elements at risk (human casualties and socio-economic losses).

5.3 MEASURES TO STOP OR DEVIATE THE PATH OF THE LANDSLIDE DEBRIS

Measures in this cathegory relate to the following cases:

a) Earth or debris flows of any type (5.3.1)¸and.
b) Toppling, rumbling or free falling rocks of various sizes (5.3.2 to 5.3.7).

They should be foreseen when the general stabilization of the landslide is not feasible from technical, environmental and financial point of view.

The basic idea of these measures is to intercept the sliding or falling material, or at least to deviate it, in order to protect existing elements at risk or points of particular interest locted downslope of a potential landslide.

5.3.1 Diversion channels

For the protection from flows it is proved that well designed (from capacity discharge point of view) channels which divert the sliding mass are by far the best method. The design of these channels must take into serious consideration the geo-morphological features of the landslide prone area and the most extreme expected meteorogical data in order to calculate the expected volume of the debris and the necessary section of the channel. Otherwise the consequences can be very serious, as can be seen in **Figure 33** which shows the effects of debris flows at Stratoni Village in Greece (Anagnostopoulos et al, 2010). Lava flow mitigation practices may be also seriously considered in the design of mitigation measures, as they present several similarities.

Figure 33: View of Stratoni Village affected by debris flows (Anagnostopoulos et al, 2010).

Several measures are available for the protection from rock falls, as described below. Examples of application are provided by Rancourt et al. (2004) and Cheer (2009)

5.3.2 Re-modelling of the slope

This can be done by making the slope gentler, or by constructing berms. According to the experience, the berms must be broad (b>4m), otherwise they can be destroyed by erosion and they are not accessible by trucks and other machines in order to perform any maintenance works on the slope (Anagnostopoulos and Georgiadis, 2009).

Figure 34: Example of eroded, small width berms (Anagnostopoulos and Georgiadis, 2009)

Moreover it has been proven that small width berms do not “work” well; they are missed by the falling rocks or they perform like springboards for the falling rocks, guiding them in bigger lateral distances (Wyllie, 2007). So, in some cases it should be beneficial to examine the possibility to cut down the berms and allow the rocks to be collected at the base of the slope.

5.3.3 Planting and vegetation on the slope

Planting on the slope acts in two ways:

1. Trees and bushes act as barriers consuming the energy of the falling rocks by their cracking
2. The surface plants act like absorbers of the energy of the rolling rocks

The most important issue in this case is whether the plants can survive for long at the slope without maintenance. The success of this method, which, needless to say, is a supplementary one, depends on the inclination of the slope, the quality and properties of the surface soil and the climatological factors affecting the area.

5.3.4 Catch trenches

Catch trenches are constructed at the foot of the slope. They must have enough width and height in order to entrap the falling rocks. Usually they are combined with a retaining wall at their end (foot hill), in order to obtain smaller width.

Figure 35: Illustrative example of catch trenches combined with retaining wall (Anagnostopoulos and Georgiadis, 2009)

The necessary width and depth of the trenches were calculated by empirically obtained graphs based on experience, like those given by Ritchie (1963). Graphs and a detailed design methodology have been presented by Pierson et al (2001). Nowadays, the necessary width and depth of the trenches can be calculated by using relevant software, as it will be presented in the following paragraphs.

5.3.5 Rockfall barriers

Typically rockfall barriers consist of a row of steel posts anchored on the slope and connected with wire nets and wire ropes. These structures are placed perpendicular to the expected trajectories of the falling rocks and their role is to block these rocks. The barriers are designed on the basis of the energy they have to absorb and the expected height of the bouncing rocks.

The main design procedure is as follows:

a) Recognition of the source areas of the falling rocks (by in situ inspection)
b) Estimation of the size of the falling rocks (by local experience and by geological investigation)
c) Consideration of the simplified slope section
d) Estimation of the bouncing properties of the slope surface
e) Calculation of the expected trajectories of the falling rocks on a stochastic basis.

f) Estimation of the best positions where stop barriers of appropriate height can be placed in order to trap the falling or rolling rocks (based on the results of the above analysis). The energy carried by the falling and bouncing rocks can be calculated in any desirable point.
g) The safe distance at which the falling rocks can travel can be easily obtained.

The design is performed by relevant software, which is commercially available (e.g. Rocfall, V4.0) or free of charge (e.g. CRSP, by Jones et al., 2000).

Figure 36: Example of investigated protection barriers during rockfall event (Volkwein et al., 2006)

Once the height and the required energy capacity of the barrier have been chosen, the design of the barrier is performed by a simple choice from a selection of barriers given by companies, which have obtained certificates for the absorption capacity of their barriers (ETAG, 2008). Only two companies are known to have obtained these certificates at the time of writing (GEOBRUGG: www.geobrugg.com and MACCAFERRI: www.maccaferri.com). This fact has led to high cost, even for simple cases.
For example in Greece the cost for a barrier of 3m high is from 1000€/m for a 250kJ barrier to 1750€/m for a 1500kJ barrier. For a 5m high barrier the cost is from 3000€/m (2000kJ) to 4000€/m (3000kJ).
In that respect, for the cases where the energy absorption demands are not so high, there is a need for more simplified design procedure, which will permit the use of much simpler and cheaper retaining solutions, based on use of commercially available materials, without the need of paying very expensive certificates. A simplified method of calculating the absorbed energy from these structures is given by JRA (1984).

Another possible solution of rock barriers is to construct earth embankments, reinforced in order to reduce land take (**Figure 37**). The results of a thorough investigation of the problem by Peila et al (2007) have led to a practical design method, which of course needs further verification. The main advantage of the method is the smaller cost but the main disadvantage is the needed space, which for several reasons is usually difficult to obtain.

Figure 37: Reinforced soil rock containment bund near Cretaz, Cogne, Italy (Officine Maccaferri)

5.3.6 Rockfall nets (or Drapery)

These nets consist from a wire net, which is anchored at the head of the slope and it has been laid on the slope. The net is reinforced by wire ropes (mainly in vertical direction). When a rock starts to fall it is guided by the net to the foot of the slope, consuming almost all its energy. The nets can be anchored on the surface of slope by small rock anchors (in case of bigger heights and bigger rocks) or they can be placed free of anchors.

The cost of these nets in Greece is 60€/m2 for the free nets and 90€/m^2 for the anchored nets. The main advantage of the method is that they can be placed and replaced easily.

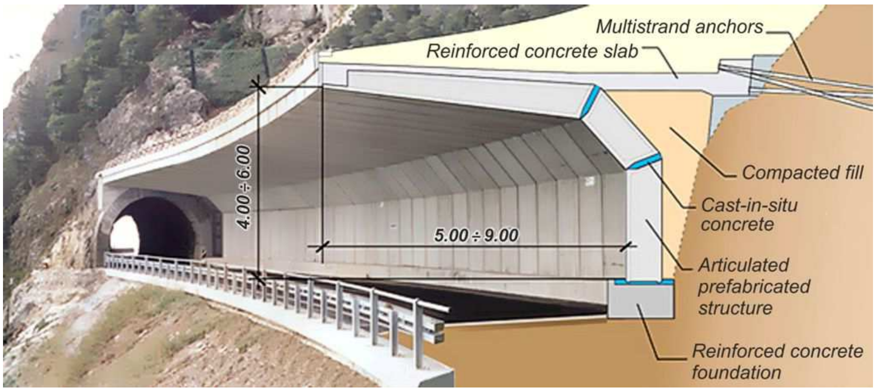
The WA-RD 612.2 Manual (Design Guidelines for wire mesh/cable nets slope protection, Muhunthan et al., 2005) provides a complete design procedure for the nets (wire net, spacing and section of wire ropes and properties of head anchors). The free software MACRO2 (2005) offers similar capabilities.

Figure 38: Wire net draping over rockfall prone rock slope (Officine Maccaferri)

5.3.7 Rock Sheds

In many cases, the protection of part of a road from falling rocks is needed. One very effective method is to cover this part with a completely new structure (usually made from reinforced concrete), properly designed to absorb the impact of the rocks and to guide them further, far from the road. Soft ground material is placed on the top of the sheds in order to reduce the effect of the impact on the structure by absorbing the energy. From the structural point of view, the sheds are designed to resist the impact with no or easily repaired damages. A very interesting synopsis of the available design methods has been presented by Yoshida et al (2007). There are many types of sheds (see JRA, 1984). They can be also designed in order to respect the environment. Although photographed in New Zealand, **Figure 39** shows a structural arrangement that is very common in Europe. **Figure 40** shows a prefabricated cantilever arrangement, recently developed and applied in Italy. The mean cost of the sheds in Switzerland, where many sheds have been constructed, is around 1.5million€/100m of shed.

Figure 39: Open rock shed (photo by R. Wright, as reported by Highland and Bobrowsky, 2008)



(a)

(b)

Figure 40: Prefabricated cantilever rock shed: (a) schematic section; (b) completed structure near Trento, Italy (www.tensiter.it)

6 MEASURES TO REDUCE THE ELEMENTS AT RISK

The temporary or permanent reduction of the number and/or value of the elements at risk is widely practiced and particularly cost effective, especially when the number of elements at risk is small in relation to the extent of the landslide and of the affected area and when it is achieved through the sustained implementation of appropriate long-term planning measures.

Ambrozic et al. (2009) distinguish between:

- Decreasing the number of vulnerable elements potentially affected by a landslide, for example by:
    - Zoning to prevent development in hazardous areas or removing existing development from hazardous areas (exclusionary zones);
    - Traffic restrictions (reduce number of vehicles).
- Decreasing the probability that vulnerable elements will both spatially and temporally intercept ground movements, e.g. by:
    - Moving non-stationary vulnerable elements to less hazardous locations;
    - Increasing awareness, detection and warning of hazards (either detected movement or trigger conditions) and subsequent avoidance (evacuation or temporary exclusion, followed by inspection before resuming normal use).

Each of these strategies can be implemented forcibly through standards and legislations or, less invasively, by means of incentives or disincentives introduced through planning.

- <u>Relocation of existing facilities</u> – Existing facilities can be completely eliminated or they can be reconverted to uses which imply a lower vulnerability to landslides.
- <u>Reduction of the specific value</u> – The average number of people and/or the value of economic activities associated with a specific element at risk can be reduced, for example by limiting the range of end uses allowed through the planning instruments. A similar result can be obtained indirectly by regulating the market, for example by introducing the duty of publicity in deeds of sale. In this case, if a given facility is located in a hazardous area and the potential buyer is made aware of this, the specific value of the facility will be reduced, although in this way only the commercial value of the facility will be decreased, not the presence of elements at risk.
- <u>Avoiding the construction of new facilities</u> – The forced relocation of existing facilities is an extremely invasive measure, potentially applicable only in the most serious situations. A more practical approach in many cases may be the implementation of a long term strategy to prevent the location of new elements within hazardous areas, either by enforcing planning limits or through policies based on incentives or disincentives. This is the least invasive approach and it can be implemented, for example, through making it compulsory to obtain insurance for elements at risk, by public information campaigns, by introducing fiscal incentives/disincentives to make it less attractive to build in hazardous areas, or by forcing the constructors to inform potential buyers of the possible risks.

A particular case of (partial) avoidance is exemplified by measures to limit the impoundment level in reservoirs, which typically combines a reduction in the specific value of the element at risk with a simultaneous reduction in the hazard, in so far as the probability of given landslides and the indirect consequences in a domino chain may depend on the depth of impoundment.

7 MEASURES TO SHARE RESIDUAL RISK

7.1 GENERAL

Among the possible strategies to manage landslide risk, techniques can be identified to increase the tolerance towards the residual risk that typically characterizes real situations even after implementing all other (technically and economically) possible mitigation measures.
Of particular interest are risk sharing arrangements, which can be either voluntary or enforced. The two main mechanisms for this are:

- Voluntary or compulsory insurance, to share the risk among a large number of people. Owners can tolerate a higher level of residual risk, since any damage that may occur would be refunded by the insurance company. Clearly, this strategy is useful especially when the elements at rik consist mainly of facilities and properties, which normally corresponds to the reactivation of slow or very slow movment.
- Compulsory systems based on taxes and public intervention in case of need, where the Public Authorithy takes on the same general role of managing shared risk as the insurance company.

The role and mechanism of insurance (private or public) is of particular interest and is discussed below addressing the question why natural hazard insurances is necessary and how insurance companies are involved in risk mitigation. Further details are presented in Annex D, together with an overview of the natural hazard insurance system in Switzerland, illustrated by three case studies and in several other countries. Reefernce here to insurance and reinsurance companies can be taken to refer equally to private and public institutions, depending on local practice. Where Public Authorithies replace private insurance companies, the face the same issues and have the same overall objective of loss reduction and efficiency.

7.2 NATURAL HAZARD INSURANCE

As noted by Smith and Petley (2009), the need for insurance arises when a risk is perceived and recurrent. The owner pays a fee (premium) that transfers the financial risk to a partner (insurer). If the premium is fixed at an appropriate rate, it will cover the eventual damage costs caused by an event, besides administrative costs and a fair compensation to the insurer. This allows the policyholder to have guarantees to enable recovery of his goods after an event. However, the existence of insurance depends on the number of insured concerned; it is necessary to have enough policyholders to be cost-effective.

Natural hazard insurances have some particularities that distinguish them from other types of insurance (car, life, fire …). Specifically, the occurrence frequency, the event size and the location, are specific parameters of natural hazard insurance (Zimmerli 2003). Some comparisons with fire insurances can be presented to illustrate these specificities (**Table 4**).

The need for anticipation and evaluation of future claims is strong for insurance companies. Nevertheless a catastrophic loss due to a major disaster, threatening the stability of insurance, is difficult to predict because major disasters are by definition at a larger scale than those which occurred routinely in previous years. In this case,it is necessary to take into account a longer statistical period to evaluate the occurrence period. Kuzak et al. (2004).

Table 4: Summary of natural disaster insurance specificities for Fire and Natural hazards. Modified after Zimmerli (2003).

Difference	**Fire**	**Natural Hazard**
Occurrence frequency	High	Low
Event size	Individual risk affected (individual building or complex of buildings)	Large part of portfolio affectd(entire districts)
Location	Low importance	High importance
Consequences		
Pricing	Minor fluctuations in the loss burden; therefore, burning cost analysis and exposure rating are sufficient	Major fluctuations in the loss burden; therefore, scientific models are required
Loss potential from single event	Low to medium	Very high
Geographical distribution	Minimal impact on losses, no accumulation control required	Major impact on losses, accumulation control important

Most natural catastrophic events affect a larger part of a portfolio, and not only a single object of the portfolio. In the case of floods and landslides, an entire district may be affected.

The spatiality parameter has an influence on the vulnerability of a portfolio. It is essential for an insurer to be sure that the type of properties insured are varied and that the geographical distribution is spread. In this way, only a part of the portfolio is concerned by a specific disaster and only a fraction of the portfolio can be destroyed by a single event (Smith and Petley 2009).

Insurance intervenes at the moment of financial compensation for damages and allows victims to rebuild after a disaster. Thus, insurance provides cash to allow rehabilitation. This can significantly improve the recovery phase of disasters at a time of extreme stress and thereby reduces disruption of normal life (Walker 2005). However, insurance companies also have a role to play before the event, by financing preventive measures (**Figure 41**).

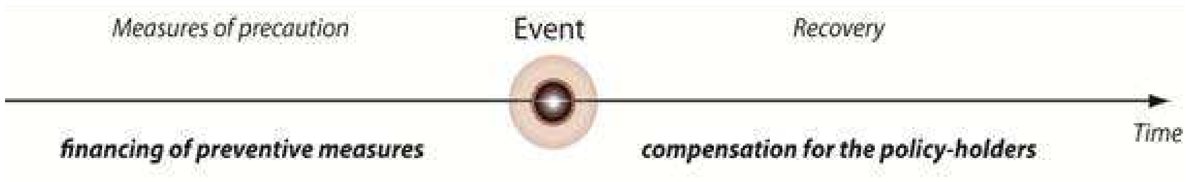


Figure 41: The insurance operates on two levels, before and after the event: financing of preventive measures and compensation for the policy-holders

Damage assessment by modeling the different components leading to financial compensation of victims is the first necessary step to a better understanding of risk. According to Khater and Kuzak (2002). These components can be described by three different modules, regardless of the kind of natural hazard: the *hazard*, the *damages* and the *loss* (**Figure 42**). These three parameters are described in the following points.

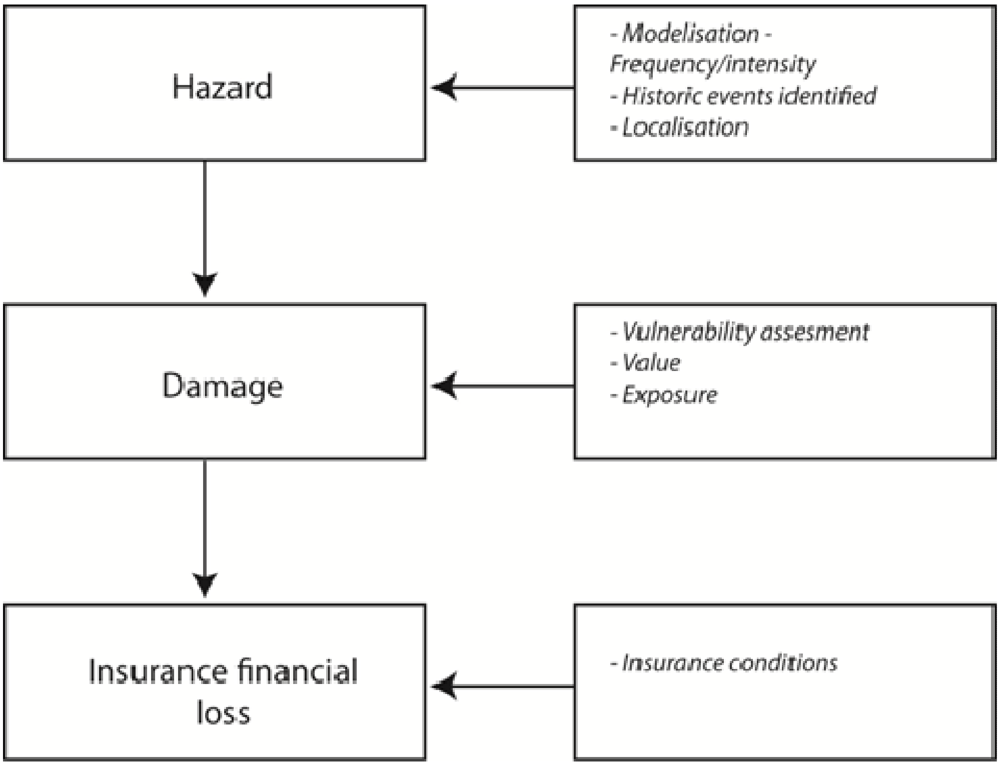


Figure 42: Component of a risk model. Modified after Khater and Kuzak (2002)

With its financial weight, the insurance industry can finance mitigation measures, participate in research about hazard assessment and reduce risk by financing protective measures.

Whatever the method used to protect properties exposed to natural hazards, a residual risk remains. This statement is demonstrated by the analysis of past events (for example BAFU 2007) where the protection measures were exceeded. This residual risk is on one hand linked to the possibility that protection measures may fail or may not work as intended. On the other hand the residual risk is linked to the possibility that the event exceeds the chosen level of protection. Many European countries, governments and insurance companies are now thinking in terms of vulnerability reduction by decreasing residual risk, since this reduction can have major consequences in financial terms.

The cost associated with natural damages has increased during the last decades worldwide; even if the damage costs have increased since the 1990's in Switzerland, the number of events is relatively stable (AEAI 2008). This increase (in economical cost) is principally a result of higher population densities, a rise in insurance density in high-risk areas and the high vulnerability of some modern materials and technologies (Zimmerli 2003).

To address this issue, insurance companies can act directly on the financial statement by increasing premiums or by decreasing allowances. Alternatively, and with significant advantages, they can act on the number of claims and/or their importance, trying to reduce the causes of the disasters; adapting buildings and thus influencing the vulnerability.

Kelman (2003) proposes an insurance system oriented towards vulnerability mitigation, called « Reverse insurance ». This system is based on an incentive to reduce vulnerability and differs radically from the systems used in major European countries. It is not the owner who pays to be insured, but the insurance (or government) who provides assistance to the insured to reduce the vulnerability of its property. It is therefore an inverse insurance system where the owner receives funding to reduce its vulnerability, while the amount of post-disaster compensation is reduced. This allows governments to better estimate the cost of disasters and it encourages locally-based vulnerability reduction and efficient innovation, although this system is not without limitations, such as the challenge of ensuring that people do use the payments for vulnerability reduction.

Financial insurance loss is determined by insurance conditions, such as deductibles, limits and total insured value (Khater and Kuzak 2002). By influencing insurance conditions, insurance companies can act directly on the financial statement by increasing premiums or by decreasing allowances.

Modeling the loss is difficult, because it has to take into account the evolution of vulnerability, land use planning, environmental conditions and the increase of population, and requires a prospective, rather than a retrospective model (Khater and Kuzak 2002).

Natural hazard insurances participate in the financial recovery after an event. Insurance companies can thus play the role of the State without altering the economy of the country. Therefore, an insurance system is a necessity to protect the local economy, while lack of insurance can discourage development in hazardous areas (Smith and Petley 2009).

By requiring obvious and defined protection goals, the insurance companies have the possibility to control the fragility of the portfolio. They may thus decide the degree of fragility of their portfolio and the "damage tolerance". Insurance companies that pay without seeking to reduce the amount of damages are not an incentive system to reduce disaster costs, because after every disaster, the owner is reimbursed.

An insurance company can transfer, against payment, part of the risk of a premium to a reinsurance company. A reinsurance company is somehow the insurance of the insurance companies. It will directly cover the damages exceeding the insurance provisions. The reinsurance companies are thus very interested to estimate the potential damages induced by natural disasters. These companies are very active in the publication of prediction of risk and natural disasters. They finance scientific studies and research work in a partnership between academia, public policy institutions and the insurance industry to lead scientific understanding of extreme events. Contrary to private insurances active at the national level, the companies of reinsurance work on the worldwide market and are consequently interested in catastrophes in a more global manner.

Hurricane "Andrew" in 1992 and more recently hurricane "Katrina" in 2005 illustrate the need for the insurance and reinsurance companies to have better natural hazard models, in order to anticipate the most important catastrophic events and to estimate the maximum potential loss.

Regarding natural hazards, it is not sufficient to anticipate the “normal” catastrophe, but it is necessary to anticipate “the worst” possible events. This is why reinsurances companies develop catastrophe risk models (Khater and Kuzak 2002).

The catastrophe risk model (**Figure 43**) combines the components leading from the risk to the loss, described above. As highlighted by these models, many possible benefits exist for insurance companies to encourage mitigation measures, as shown by Kunreuther et al. (2004):

a) Reducing direct losses: Mitigation measures can avoid physical damages caused by the disaster to insured infrastructures as well as the loss of lives. For example for rock falls, building a reinforced wall can avoid building collapses and save lives.
b) Reducing indirect losses: This concerns the loss induced by the catastrophe but not directly to the infrastructure. This can be a long-term loss, for example a business interruption, causing a loss other than the direct loss.
c) Reducing losses to neighboring structures: A mitigation measure can avoid damage to other infrastructures, without having been designed for the neighborhood. For example, a building collapse can damage other buildings that would have been left standing otherwise. Mitigation measures that avoid the collapse reduce also the loss to neighboring structures.
d) Reducing financial costs from catastrophic losses: the mitigation measure can reduce the catastrophic losses and thus avoid the recourse to public finance envisaged in the case of great catastrophes exceeding the financial capacities of the private insurers.

With their financial strength, insurance companies have the possibility to influence the economic losses due to natural hazards. This can be done either by reducing allowances, through incentives to reduce the vulnerability of properties, through research or by directly influencing the owner. The reduction of allowances to the policy-holder does not seem to be the most optimal way, because this benefits only the insurer and not the policy-holder.

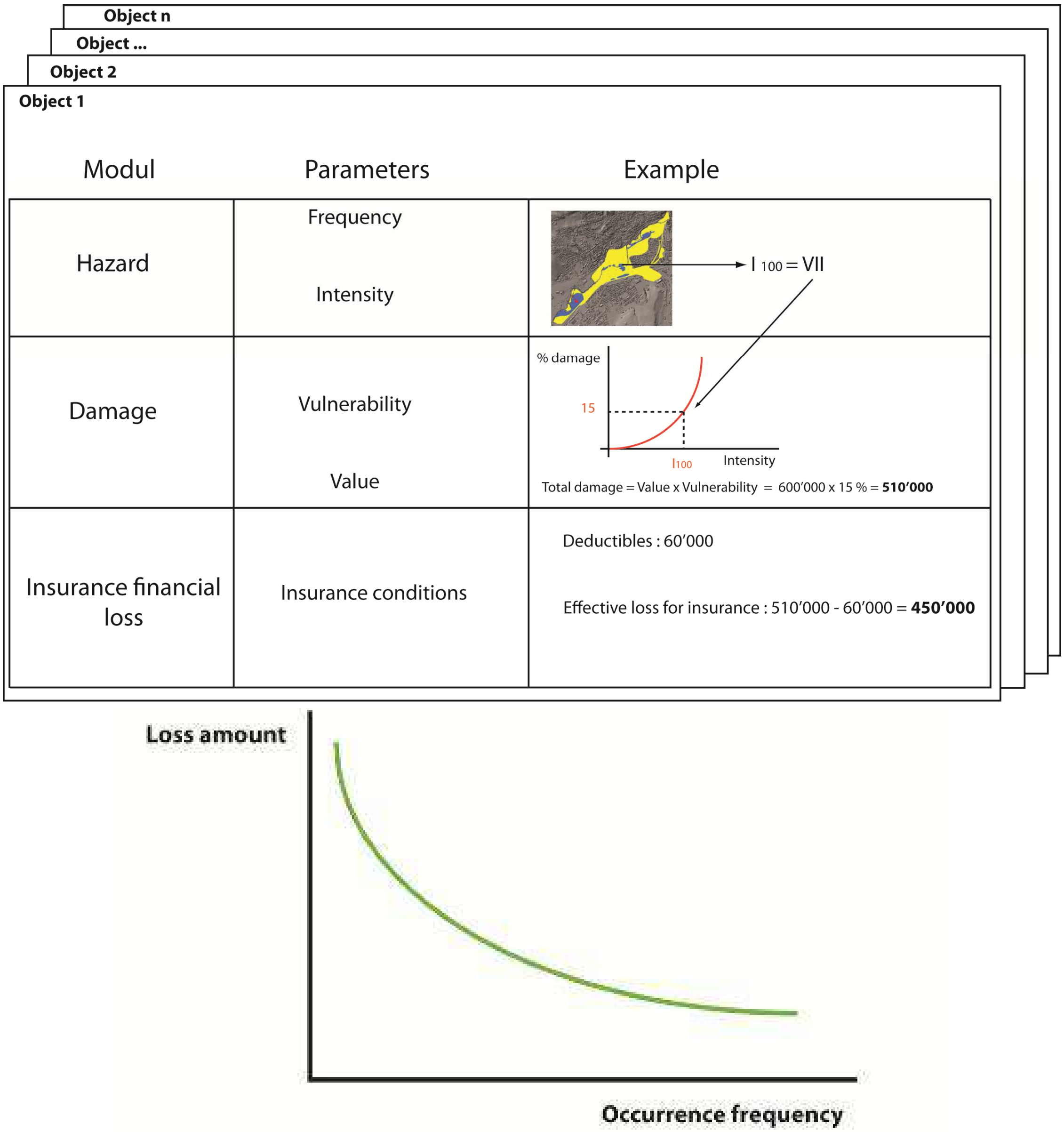


Figure 43: Loss amount vs. frequency of occurrence. Summing the economic losses for all the objects gives a model of a catastrophic loss. Modified after Khater and Kuzak (2002) and Zimmerli (2003).

Object vulnerability reduction will certainly be a challenge for the coming decades. With the current trends of ever increasing damage costs and the prospect of an increase of natural disasters induced by global warming, many institutions will have to take into account the fragility of exposed objects. Indeed, the vulnerability of a given object has a huge impact on the final amount of damages. Reducing the vulnerability of a person's property is important and beneficial to decrease the amount of damage.

By focusing on this research area, particularly through laboratory research or partnership with the scientific community, insurance companies seem to have anticipated this problem.

However, vulnerability is not always taken into account by owners; even when they are aware of the danger in which their property lies. Indeed, the systematic reimbursement of damage (or even only the expectation of systematic reimbursement) does not encourage owners to take initiatives to reduce vulnerability, even though simple measures to reduce vulnerability could be effective in most cases.

According to Munichre (1997), motivation through financial incentives "*has already proved to be one of the most effective ways of encouraging the owner to take precautions. The best approach is to make sure that clients retain an adequate proportion of the risk themselves, especially by introducing substantial deductibles*".

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ANNEX A

MITIGATION THROUGH REDUCTION OF HAZARD

FACT SHEETS

MITIGATION THROUGH REDUCTION OF HAZARD

FACT SHEET 1

SURFACE PROTECTION - EROSION CONTROL

MITIGATION THROUGH REDUCTION OF HAZARD

1 SURFACE PROTECTION - EROSION CONTROL

1.0 GENERAL

Basic principles

The main goal of erosion control is to protect the face of the slope and to strengthen subsurface parts, typically by interlocking soil particles with a complex matrix of roots. The stability of slopes is dependent on the ratio of driving forces and the strength of the soil-root system. The weight of vegetation growing on the slope accounts for a part of the driving forces but the roots add to the shear strength of the soil. Vegetation also intercepts rain, by reducing its impact energy and preventing splash erosion and slowing down runoff. Figure 1 shows the decrease of erosion rate as the soil is covered by vegetation.

Vegetation also changes the pore pressure in the soil via the evapotranspiration process (Morgan & Rickson, 1995). This process decreases the pore pressure and increases the effective stresses in the soil, which also improves the shear strength (Figure 2). But unfortunately, in temperate European climates, the season of peak water demand by vegetation (summer) is out of phase with the season of greatest rainfall (winter) (Smethurs et al., 2006 and Thielen et al., 2011).

Table 1: The effects of vegetation on the stability of slopes (after Wu, 1995).

Process	*Type*	*Effect on stability*
Increase of permeability, infiltration and pore pressure	Hydrologic	Negative
Increase in interception and evapotranspiration and decrease in pore pressure	Hydrologic	Positive
Increase of weight on the slope	Mechanical	Negative
Increase in wind resistance	Mechanical	Negative
Reinforcing the soil by roots	Mechanical	Positive

Initial conditions for bio-engineering measures are usually rather unfavourable. The area to be stabilised is often barren, partly unstable and erosive processes abound (Graf & Gerber 1997; Graf et al. 2003).

The long-term effects of bio-engineering stabilisation methods depend on site characteristics, slope failure processes and the technical and biological measures employed (Stokes et al. 2007). Detailed analysis of the stability of the slope is necessary to determine the suitable stabilising method. One of the greatest uncertainities concerns the depth of the potential sliding surface and the measures have to be chosen accordingly

Slope stability and the efficiency of stabilising measures are usually influenced not only by soil mechanics but also by hydrological factors and hydraulics. The combined effects are rather complex and are often responsible for failure (Boll, 1997). Surface erosion and landslides are usually long-term processes (over some decades and more) and stabilising measures are required to have a correspondingly long lifespan. The bearing capacity and functionality of supporting structures are likely to become critical in the course of time, and biological measures may fail to prosper. Periodical site inspections are therefore necessary to plan maintenance and/or replacements properly. Knowledge about the development and long-term behaviour of joint technical and biological methods is indispensible (Pastorok et al., 1997; Anand & Desrochers, 2004.

Quantification of the stabilising effects of vegetation (based on Frei, 2009)

In recent years, several studies have been performed to describe vegetation effects quantitatively. According to Simon & Collison (2002), root-permeated soil makes up a *composite* material that has an enhanced strength. In general, soil can resist against compression stress, but can hardly resist against tensile stress. The fibrous roots of trees and herbaceous plants, on the other hand, can resist against tensile stress, but hardly against compression stress (Nilaweera & Nutalaya, 1999). However, to implement this analysis method in practice, there are restrictions with respect to the root distribution. Usually, only man-made brush layers achieve this condition. Therefore, this model is inappropriate to provide a generalised representation of vegetation effects (Frei, 2009).

If a slip plane is penetrated by roots, they can be included in stability analyses comparable to ground anchors operating with a tieback function. But it requires careful attention to determining the exact root distribution, as well as the pull-out resistance of the different root classes to be able to quantify any anchoring effect of roots. Therefore, this model is inappropriate

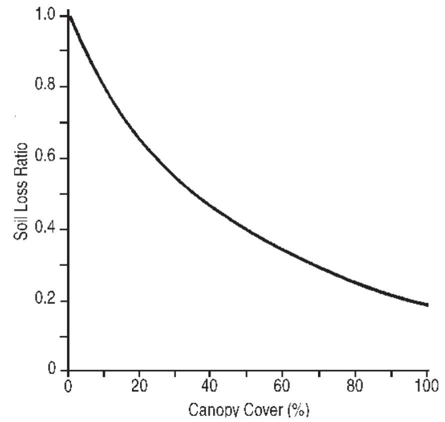

Figure 1: Change in erosion rate due to increasing vegetative cover
(*soil loss ratio: soil losses measured on cropped area / corresponding loss from clean-tilled area, canopy cover: the percentage of planted area*).
(source: Coppin and Richards, 1990)

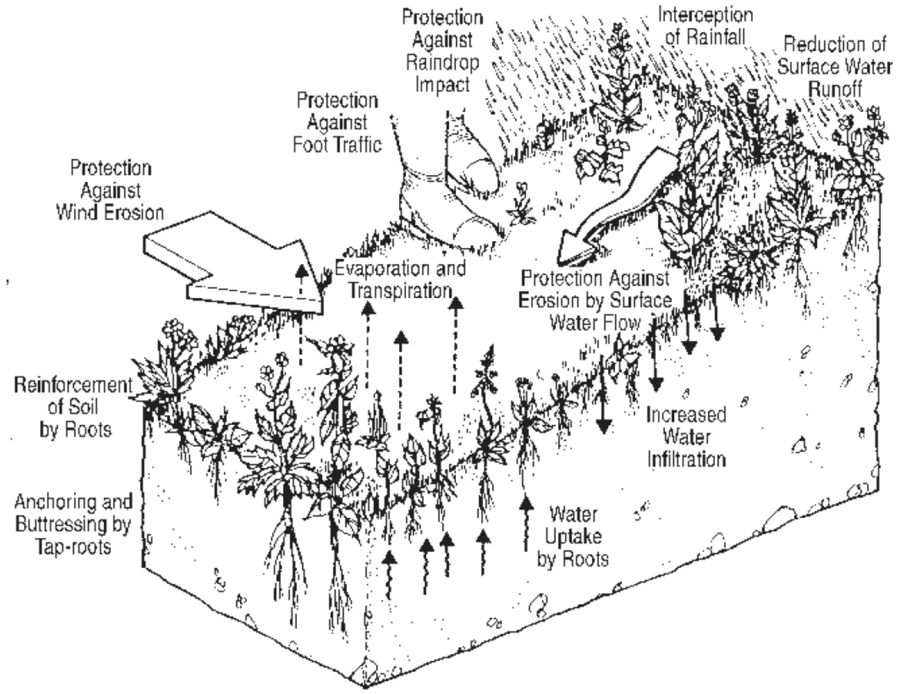

Figure 2: Some influences of vegetation on the soil.
(source: Coppin and Richards, 1990)

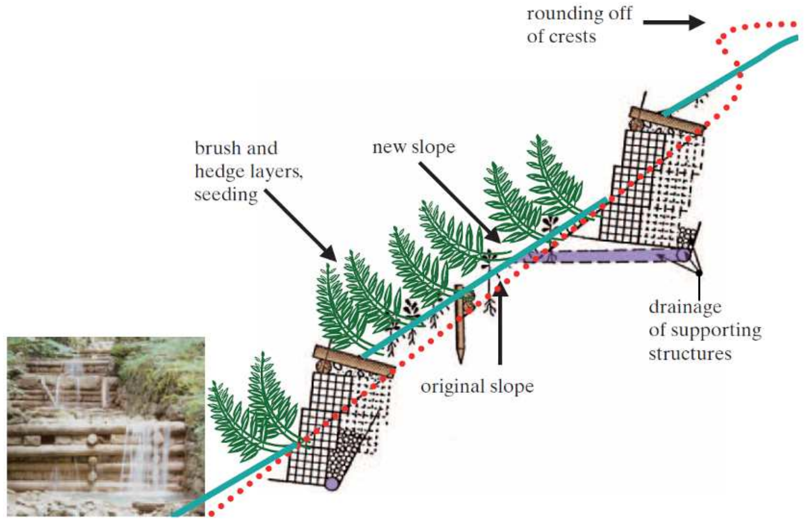

Figure 3: Retaining walls and biological measures stabilise a slope against surface erosion and shallow landslides. If the foot of the slope connects with a torrent, check dams in the latter are a prerequisite. The supporting structures (gabions in this example) are each protected against rockfall by a log grid on top of them (source: Boll et al. 2009).

MITIGATION THROUGH REDUCTION OF HAZARD

1 SURFACE PROTECTION - EROSION CONTROL

1.0 GENERAL

A further possibility is to assign vegetation effects to the soil shear strength directly. In doing so, two approaches can be taken: those that immediately measure the shear strength and methods that assign vegetation effects to the shear strength parameters. The direct measurement of the shear strength of root permeated soils can be performed by means of a direct shear apparatus, as described in Waldron et al. (1983), Wu (1984) and Tobias (1992). According to Boll & Graf (2001), the disadvantage of this method is that the failure plane is predefined (by the apparatus) and that the result obtained by such field tests represents only a pure shear resistance (analogous to a ring-shear test to determine the undrained shear strength of a fine grained soil). The influence of shear pane undulation or any other layering or discountinuities my not be taken into account. As a consequence, such a value is not usually appropriate for classical stability analyses. If the shear strength is written according to the Mohr-Coulomb failure criterion (Terzaghi & Peck 1967), then it can be directly integrated in stability analyses. Wu et al. (1979) as well as Wu (1984) assign any vegetation effects to the soil cohesion, by introducing an additional cohesion component due to the root reinforcement (c_r). Variations in mechanical reinforcement at the root-zone-scale are particularly important for small and shallow landslides with areas of 10 to 2000 m^2 (Reneau & Dietrich, 1987). Moreover, complexities arising from the distribution of root sizes and details of root-soil mechanical reinforcement also demonstrate that application of a uniform cohesion term may represent an oversimplified picture that could overlook susceptibilities emerging when a more complete stress–strain relationship of root systems and characteristics of their distribution are included in calculations of slope stability (Schwartz et al., 2010).

Boll & Graf (2001) regard this additional parameter as simple to determine, but it represents the conditions in superficial soil layers far less optimally than the stress-dependent expression in the frictional component of the Mohr-Coulomb notation. Since the roots exert a form of prestress on the surrounding soil grains, this is analogous to increasing the contact stresses which will contribute to additional shear strength through the modification of frictional resistance. Therefore, adding an additional component to the friction angle would represent the mobilised shear resistance under a greater range of valid stress conditions near the surface. It was postulated that, it would be more convenient for designers to describe the resistance mobilised and hence the stability in the vegetation influenced superficial soil area. However, there are no suitable models available yet (Frei, 2009).

Schwartz et al. (2010) reviewed the primary geometrical and mechanical properties of root systems and their function in stabilizing the soil mass. They considered the stress–strain relationships for a bundle of roots using the formalism of the fibre bundle model (FBM) that clumps the effects of roots together and offers a natural means for upscaling mechanical behaviour of root systems. They proposed an extension of the FBM, considering key root and soil parameters such as root diameter distribution, tortuosity, soil type, soil moisture and friction between soil and root surface. The spatial distribution of root mechanical reinforcement around a single tree is computed from root diameter and density distributions and is based properties that can be measured easily. The distribution of root reinforcement for a stand of trees was obtained from spatial and mechanical superposition of individual tree values with respect to their positions on a hillslope. This method has been applied to a full scale rainfall triggering test (Springman et al., 2009) and the results of simulated falire zone (Schwartz, 2010) shows good agreemets with the real failure wedge (Askarinejad et al., 2010).

Hydroseeding (based on BMP Handbook)

Hydroseeding typically consists of applying a mixture of wood fibre, seed, fertilizer, and stabilizing emulsion with hydromulch equipment, to temporarily protect exposed soils from erosion by water and wind (Figure 5). Hydroseeding is suitable for areas requiring temporary protection until permanent stabilization is established.

<u>Limitations</u>

- Hydroseeding may be used alone only when there is sufficient time in the season to ensure adequate vegetation establishment and coverage to provide adequate erosion control. Otherwise, hydroseeding must be used in conjunction with mulching (i.e., straw mulch).
- Steep slopes are difficult to protect with temporary seeding;
- Temporary seeding may not be appropriate in dry periods without supplemental irrigation.
- Temporary vegetation may have to be removed before permanent vegetation is applied

<u>Inspection and maintenance</u>

- Where seeds fail to germinate, or they germinate and die, the area must be re-seeded, fertilized, and mulched within the planting season, using not less than half the original application rates

Figure 4: Schwandrübi scree slope. Left: in 1978 prior to the application of joint technical (gabions) and biological (*Alnus incana*, *Salix purpurea*, hydroseeding) measures conducted during 1981 and 1982 and right: in 2005 after the heavy rainstorm event in August. (source: left Forestry Service of Nidwalden; right WSL)

Figure 5: Aerial Hydroseeding (http://www.ericksonaircrane.com/hydroseeding.php)

MITIGATION THROUGH REDUCTION OF HAZARD

1 SURFACE PROTECTION - EROSION CONTROL

1.0 GENERAL

- Irrigation systems, if applicable, should be inspected daily while in use to identify system malfunctions and line breaks. When line breaks are detected, the system must be shut down immediately and breaks repaired before the system is put back into operation;
- Irrigation systems shall be inspected for complete coverage and adjusted as needed to maintain complete coverage.

Turf reinforcement mats (based on www.urbancreeks.org)

The use of Erosion control blankets are considered as temporarily stabilisation method and protect disturbed soil from raindrop impact and surface erosion. They increase infiltration, and conserve soil moisture. Mulching with erosion control blankets will increase the germination rates for grasses and legumes and promote vegetation establishment.

Erosion control blankets are used on slopes and disturbed soils where mulch must be anchored. They are applied for steep slopes, generally steeper than 3:1, and slopes where erosion hazard is high. Their use is especially appropriate for critical slopes adjacent to sensitive areas, such as streams and wetlands, and disturbed soil areas, where planting is likely to be slow in providing adequate protective cover. Establishing vegetation in channels or on slopes may require additional measures beyond seeding and straw mulching.

Materials

Erosion control blankets are generally a machine produced mat of organic, biodegradable mulch such as straw, curled wood fiber (excelsior), coconut fibre or a combination thereof, evenly distributed on, or between photodegradable polypropylene or biodegradable natural fibre netting. Synthetic erosion control blankets are a machine-produced mat of ultraviolet stabilised synthetic fibres and filaments. The netting and mulch material are stitched to ensure integrity and the blankets are provided in rolls for ease of handling and installation.

Advantages

Erosion control blankets can provide immediate soil surface stabilisation. Even if herbaceous vegetation does not grow, the blankets will provide excellent protection for at least one season. Woody cuttings such as stakes, wattles and fascines may be used with erosion control blankets and geotextiles.

Disadvantages

The slopes must be uniform and relatively smooth before installation to ensure complete contact with the soil. The associated labour cost may be higher.

Bushes and trees)

Bio-engineering systems are usually established by conventional seeding of the plants or live planting (Morgan & Rickson, 1995). The main goal of these systems are reducing surface erosion and reinforcing the soil. The construction methods used mainly rooted cuttings and these are installed in different configurations. The effectiveness of this system as soil reinforcement depends on the depth at which cuttings can be placed and the depth to which the roots can penetrate. Soil reinforcement systems by bushes and trees are described by Leiser (1982), Copping & Richards (1990). The growth rate of roots is related to the volume of the cuttings and some guides on choice and preparation of cuttings have been given by Gray & Leiser (1982) and Schiechtl (1980). The species should have a root system that penetrates to the required depth to creat favourable conditions for stability. In humid regions, bushes and trees with high transpiration would be more effective in decreasing of water content. The characteristics of the plants are summarized in Table 2.

Table 2: Characteristics of plant groups, (after Morgan & Rickson, 1995).

Ecological criteria	*Resistance to drought, salt, and temperature extremes*
Growth characteristics	Ease of propagation, growth rate requires consideration of cutting material, humidity, temperature, light, soil type and time of propagation
Engineering properties	Root strength, depth and diameter of root systems, water use

Wherever feasible, native vegetation is preferred and the succession from pioneer to climax bush or tree in the site environment, primarily climate and soil type and moisture, should be considered (Morgan & Rickson, 1995, Gray & Leiser 1982, Schiechtl 1980).

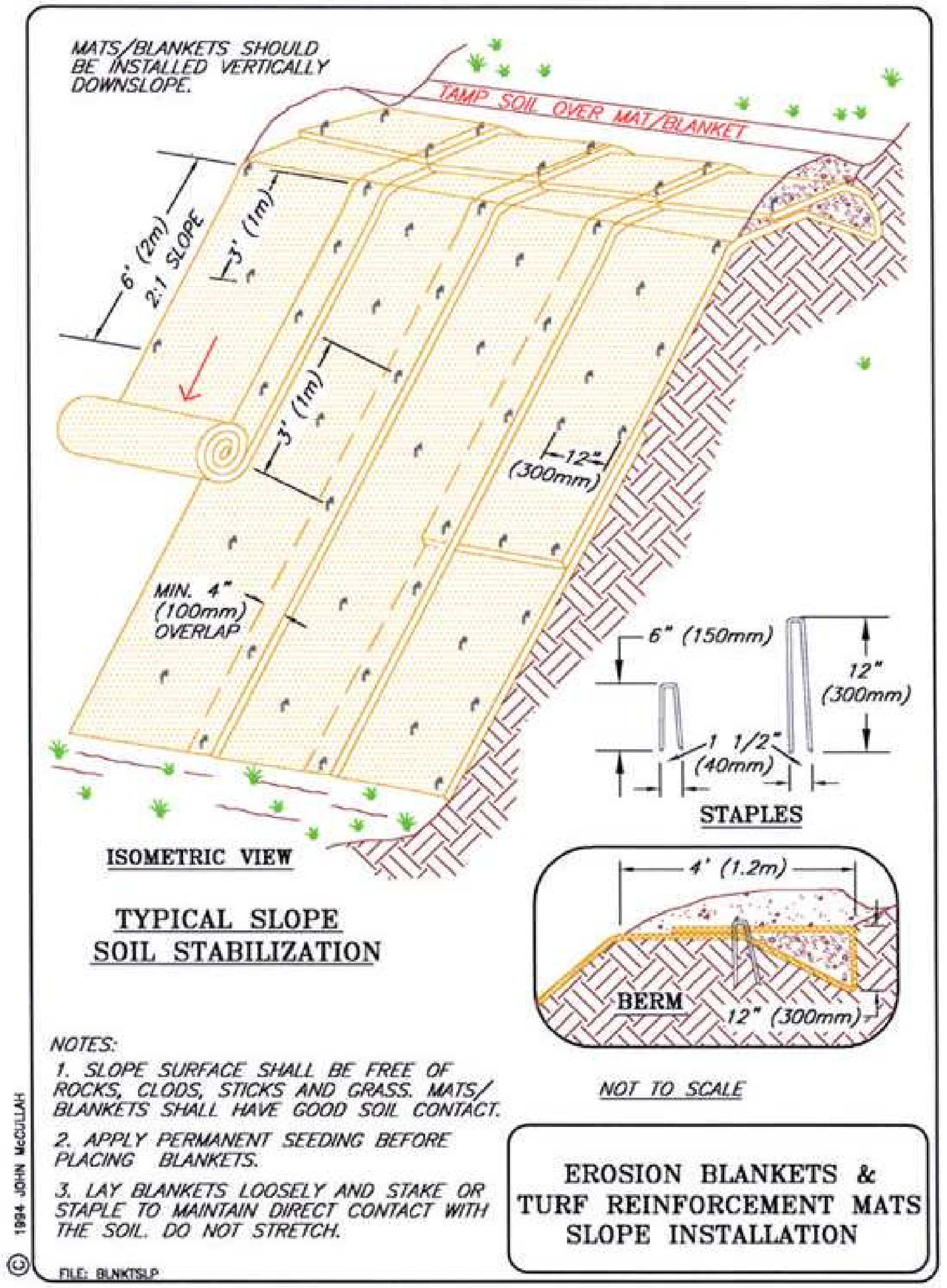

Figure 6: Typical drawing: Erosion Control Blankets for soil slope stabilization (http://www.urbancreeks.org/Erosion%20Control%20Blankets.pdf)

MITIGATION THROUGH REDUCTION OF HAZARD

1 SURFACE PROTECTION - EROSION CONTROL

1.0 GENERAL

Fascines/Brush (based on www.ohiodnr.com/water/pubs)
Fascines or brush mattresses are particularly suited for especially erosion-prone areas and in cases where differing substrates (e.g. topsoil onto raw soil) are put onto slopes without being sufficiently interlocked and are most often used to stabilize fairly long slopes.
Fascines are made of up bundles of thin live cuttings of willow or red-osier dogwood. Live fascines (LF) and inert fascines (IF) are sausage-shaped bundle structures made from cuttings of living woody plant material, 20-25 cm in diameter and 1-6 m in length. In the LF, the cut branches are expected to grow producing roots and top growth, (performing additional soil reinforcement via the roots and surface protection via the top growth) (Gerstgraser, 1998). Fascines are placed in grooves parallel to the slope and are fastened with wooden stakes (Figure 7 & 8). The plant-filled trenches break up the length of the bank face, shortening each slope segment and reducing the energy available for erosion. The lines of vegetation placed parallel to the contour of the shore can break up the erosive force of small waves since the plants grow in lines perpendicular to the source of energy.
The IF is not intended to grow, but can be used to protect the toe of the streambank while other vegetation becomes established (Figure 9) (Sotir & Fischenich, 2001).
For brush mattresses and hedge mattresses, dormant plants or plant cuttings are laid crisscross onto 50-200 cm wide berms and are then covered with soil and carefully compacted (Figure 10) (Allen & Fischenich, 2001).

Operation and maintenance (based on Sotir & Fischenich, 2001 and Allen & Fischenich, 2001)
The stream and corresponding parameters like velocity, flood frequency, flood stage, timing, and future planned use governs the operation and maintenance program. As with any live plant, health, growth and form need to be evaluated periodically to assure its continued function. Repair of the system may be required until the vegetation becomes well-established. Successful plants will grow vigorously and spread their roots into the surrounding substrate. If animal or human damage is evident, preventative measures, such as exclosures, may be required. Such exclosures, especially for woody plants, may only need to be used until the vegetation is well-established. Inspections are needed after high water events during the first year and once a year thereafter.

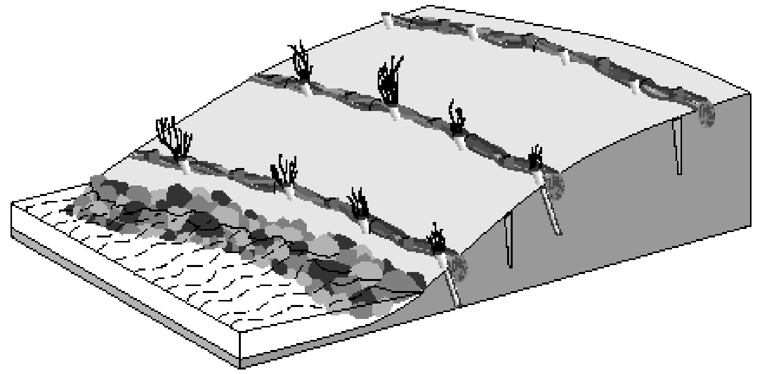

Figure 7. live fascine structure (www.nrcs.usda.gov/technical/stream_restoration)

Figure 8. Installing a live fascine structure (Sotir & Fischenich, 2001)

Figure 9. Installing an inert fascine structure (Sotir & Fischenich, 2001)

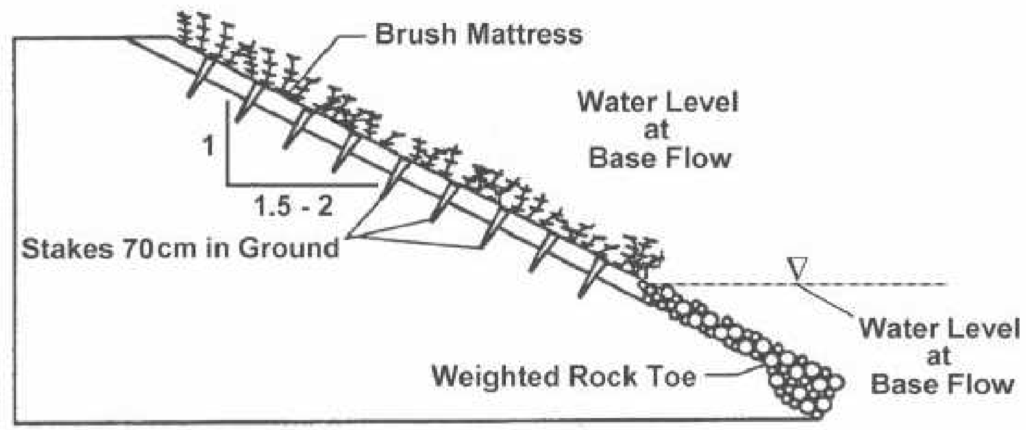

Figure 10. Profile view of brush mattress with rock toe; note relationship to water level at basal flow and that live brush is not permanently flooded (Allen & Fischenich, 2001)

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1 SURFACE PROTECTION - EROSION CONTROL

1.0 GENERAL

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MITIGATION THROUGH REDUCTION OF HAZARD

FACT SHEET 2

MODIFYING THE SLOPE GEOMETRY/MASS DISTRIBUTION

MITIGATION THROUGH REDUCTION OF HAZARD

2 MODIFYING THE SLOPE GEOMETRY/MASS DISTRIBUTION

2.0 GENERAL

Basic principles and physical process

Total or partial removal of the actually or potentially unstable mass, toe weighting and more generally modification to the geometry and/or mass distribution of slopes are widely used techniques to mitigate the hazard, and to some extent the consequences, of landsliding.
The principles underlaying the complete removal of the potentially or actually unstable mass, be it in soil or rock, including “scaling” otherwise stable rock slopes to remove rockfall hazard, are self explanatory.
Reprofiling, unloading by excavation or by partial replacement with lightweight fill at the head and loading at the toe with fill and/or gravity structures operate on the principle of modifying the balance between driving and resisting forces.
This technique is potentially effective in all materials, except those susceptible to weakening instability or liquefaction.
As also summarized for example by Hutchinson (1977), cuts and fills appear to be most effective as a hazard mitigation measure when applied to deep-seated landslides, where the slip surface tends to fall steeply at the head and rise appreciably in the region of the toe (rotational and pseudo-rotational slides). Clearly, the effect of a given cut or fill on the overall factor of safety depends on the size of the landslide being treated.
While localized mitigation by cuts and fills may prove very effective in dealing with the specific failure surface for which they have been designed, it is important to ensure that they do not cause instability themselves, either locally or to the rest of the slope outside the original landslide being addressed. It is important to note also that in some cases, especially in long translational slides, they may be quite ineffective against almost equally serious landslides involving only a portion of the slide, as shown for example slide a-b-d overriding the fill placed to stabilize the slide a-b-c in Figure 1 (Hutchinson, 1977).

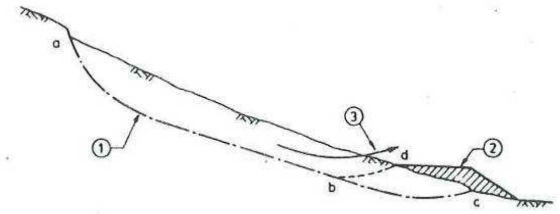
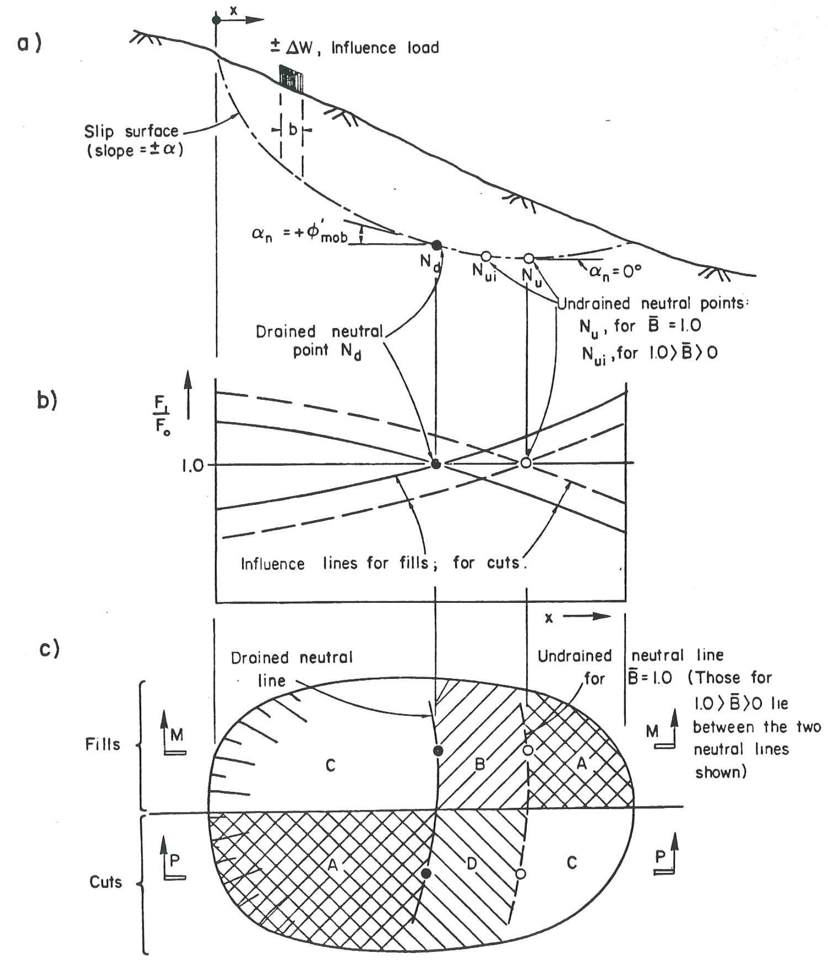


Figure 1: Translational slide stabilized by toe fill and the danger of potential over-rider slides.
1) slip surface; 2) toe fill; 3) over-rider slide (after Hutchuinson, 1977)

The neutral line concept and its application

The correct positioning of cuts and fills on slopes is a great importance, as is proper drainage. The respective merits of removing the head of an actual or potential slide, flattening the slope uniformly or benching it, or of building a berm at its toe have been discussed extensively in the literature.
The efficacy of a corrective cut or fill is controlled by its location, weight and shape and the characteristics of the actual or potential landslide to be treated. In order to assist the design, Hutchinson (1977, 1984) proposesd the “neutral line” concept to evaluate the relative merits of performing cuts and/or fills at different locations in the slope.
“Influence lines” can be drawn to represent how the factor of safety for sliding along a particular failure surface is modified by an “influence load” moving across the slope. Fills tend to decrease the existing factor of safety F_o when they are placed close to the head of the slide and decrease it when they are placed close to the toe. Of particular interest is the point where $\Delta F = 0$, termed the “neutral line”, which forms the boundary between areas where a fill or cut would improve stability and areas where the reverse applies.
For circular slip surfaces, in undrained conditions the position of the neutral point is vertically below the centre of the slip surface, where its inclination α is equal to zero, whilst in drained conditions it is shifted uphill, where the slope of the failure surface has the same value as the mobilized friction angle (Figure 2). For non-circular slip surfaces the neutral line will widen to become a neutral zone if the failure surface has a planar section with the same inclination as the mobilized angle of friction.
The “neutral line” concept can be particularly valuable in the early stages of planning and design, for example when it comes to identifying the optimum route of a road through an existing landslide or to make preliminary quantitative estimates of the improvement in factor of safety produced by a give design being considered. A final check should always be carried out by conventional analysis.



ZONE	FILL		CUT	
	Short Term	Long Term	Short Term	Long Term
A	↑	↑	↑	↑
B	↓	↑	—	—
C	↓	↓	↓	↓
D	—	—	↑	↓

Figure 2: Neutral lines and the effect on slope stability of fills and cuts in the short term and the long term (after Hutchinson, 1984)

MITIGATION THROUGH REDUCTION OF HAZARD

2 MODIFYING THE SLOPE GEOMETRY/MASS DISTRIBUTION

2.0 GENERAL

Slope stability analysis under static conditions

Soils and/or weathered heavily fractured rock masses

In soil and/or weathered heavily fractured rock mass, not susceptible to weakening instability or liquefaction and for which the "continuum-equivalent model" can be reasonably applied, the stability of slopes is routinely evaluated according to the Limit Equilibrium Method in 2D conditions (see for example Bishop, 1955; Morgenstern and Price, 1965; Janbu, 1968; etc.). Three-dimensional effects are generally neglected or taken into account by introducing "some" correction factors in the safety factors calculated in 2D conditions (see for example Azzouz and Baligh, 1983; Turner and Schuster, 1996).

- Three-dimensional stability analyses with simplified methods (see for example: Chen and Chameau, 1982; Leschinsky and Baker, 1986; Hungr, 1987; Gens et al., 1988; Hungr et al., 1989; Leschinsky and Huang, 1992; Lam and Fredlund, 1993; Stark and Eid, 1998, etc.) are rarely performed for the following reasons:The majority of work on this subject suggests that the 2D safety factor is conservative (see for example Hutchinson and Sarma, 1985; Hungr, 1987).
- Even when 3D analyses may be justified on geometric grounds (morphology and macrostructure), the available simplified methods, being often based on extrapolations of 2D methods of slices to 3D methods of columns, involve numerous assumptions related to side forces that are non easily justified;
- In cases where the critical surface is unknown it is difficult to set up general algorithms that would search for the critical surface, especially in cases where it may not be spherical.

However, the assumption that 2D analyses lead to conservative safety factors need some qualification (see for example Griffiths and Marquez, 2007):

- A conservative result will be obtained only if the most pessimistic section in the 3D problem is selected for 2D analyses (Duncan, 1996); in a slope that contains layering and strength variability this "most pessimistic" section may not be intuitively obvious.
- An unusual combination of soil properties and geometry could lead to a 3D mechanism that is more critical. Bromhead and Martin (2004) argued that some landslide configurations with highly variable cross-sections could lead to failure modes in which the 3D mechanism was the most critical.
- Other investigators have indicated more critical 3D safety factors (Chen and Chameau, 1982; Seed et al., 1990), although this remains a controversial topic.
- The corollary of a conservative 2D slope stability analysis is that "back analyses" of a failed slope will lead to an "unconservative" overestimation of the soil strength (Cantoni and Rocchi, 1999; Arellano and Stark, 2000).

The hypothesys on which Limit Equilibrium Methods are based are such that these methods can be applied reliably only in the case of potential reactivation of pre-existing slides; only in these cases can the geometry, the porewater pressure regime and the "operational" strength along the pre-existing shear surface be known with an acceptable degree of precision.

First time slides involve much more complex processes which determine the amount and distribution of strengths inside the potentially unstable soil/rock mass; the "operational" values at incipient failure can be only estimated on empirical/experience basis, with a relatively high degree of uncertainties.

A higher degree of realism could be reached only by finite element (FE) approach implemented taking into account appropriate mechanical and hydraulic boundary conditions, the porous (two or three phases) nature of the materials and these are characterized using appropriate constitutive laws (elasto-plastic, elasto-viscoplastic) selected on the basis of their geotechnical behaviour as reflected by experimental tests in situ and in laboratory (see for example Potts and Zdravkovic, 2001; Vaughan et al., 2004 and section 2 and 4 of the D1.1).

Thanks to the remarkable increase in computational power in recent years, meaningful 3D analyses may also be performed (see for example Chen et al., 2005; Griffiths and Marquez, 2007).

In all cases, the results of the analytic approach are very sensitive to the piezometric regime considered in the analysis and its variation with time in relation to hydrogeological and metereological conditions; the piezometric regime and its variation with time is seldom known with any great detail and is often much more complex than can be modeled in practice, especially for unsaturated materials.

It is therefore advisable always to calibrate the method and results of any analysis with well documented, representative case histories, where available.

Rock masses where stability is governed by discontinuities

The stability of rock masses is governed by the number, orientation and characteristics of discontinuities; in these cases, a continuoum-equivalent model may not reflect the behaviour of the rock mass and specific methods of analysis must be applied.

Single blocks may be analyzed by simplified limit equilibrium methods which consider sliding on one or more discontinuities which define a kinematically admissible mechanism (Hoek and Bray, 1981; Moore, 1986; Giani, 1992; Norrish and Wyllie, 1996), as follows:

- Planar failure, governed by a single discontinuity surface dipping out of a slope face;
- Wedge failures, governed by two discontinuities with a line of intersection that is inclined out of the slope face;
- Toppling failures, involving slabs or columns defined by discontinuities that dip steeply into the slope face.

The anlysis of toppling failures has been investigated by several researchers, including Goodman andd Bray (1976), Hittinger (1978), Hoek and Bray (1981), Choquet and Tanon (1985) and Wyllie (1992). Reference may be made to Norrish and Wyllie (1996) for a summary and discussion of applicable methods.

More complex situations, including those where kinematically admissible mechanism are not present, can be analyzed by the discrete elemnt method (Cundall, 1987; Lorig et al., 1991), which does not require a prescribed failure surface, and determines by iterative calculation the demarcation between stable and unstable blocks. The rock mass is modeled as individual bloks that can undergo large relative rotation and/or displacement, generating changes to the interaction forces between blocks. The solution scheme is sxplicit in the time domain and can thus simulate progressive failure.

Where stability is governed by the discontinuities in an otherwise competent rock mass, the main potential sources of errors in the analysis relate to the accuracy with which it is possible to determine and model the actual geometry of the discontinuities, their "operational" mechanical characteristics and the water pressure distribution in the rock mass and their variation with time, both as a result of changes in the boundary conditions and as a result of the response of the system to the environmental conditions. It is therefore advisable always to calibrate the method and results of any analysis with well documented, representative case histories, where available.

Slope stability analysis under seismic conditions

For the analysis of slope stability in seismic conditions, reference may be made to the three different approaches listed below in increasing order of complexity (Kramer, 1996):

- Static equivalent analysis (Seed, 1979; Marcuson, 1981; Hines-Griffin and Franklin, 1984); analytical solution are available for translational slides (Hadj-Hamou and Kavazanjian, 1985).
- Newmark type of analysis; (Newmark, 1965; Sarma, 1975; Franklin and Chan, 1977; Makdisi and Seed, 1978; Constantinou and Gazetas, 1984; Lin and Whitman, 1986; Faccioli et al., 1987; Ambraseys and Menu, 1988; Yegian et al., 1991; Crespellani et al., 1996; Crespellani et al., 1998).
- FEM analysis (Prevost et al. 1985; Griffiths and Prevost, 1988; Finn, 1988; Elgamal et al., 1990; Succarieh et al., 1991; Ktenidou and Pitilakis, 2007).

When applying the methods listed above the following aspects shall be clearly borne in mind:

- Static equivalent analyses are normally carried out with the use of limit equilibrium methods. The results are critically dependent both on the selected value of the operational shear strength and on the selected value of the pesudostatic seismic coefficient kh. In recognition of the fact that the actual slopes are not rigid and that the peak acceleration exists for only a very short time and varies across the landslide mass due to phase differences, the pseudostatic seismic coefficients used in practice generally correspond to acceleration values well below the peak ground acceleration amax. There are no hard and fast rules for selection of a pseudostatic seismic coefficient; however, it is clear that it should be some fraction of the actual anticipated level of peak acceleration in the failure mass (including amplification or deamplification effects).
- In Newmark type analyses, the evaluation of the yield pseudostatic seismic coefficient khy is normally carried out with the use of limit equilibrium methods; its value is critically dependent on the selected value of the operational shear strength. The earthquake-induced slope displacements estimated by this approach are very sensitive to the value of khy; small differences in khy can produce large variations in predicted slope displacements. Furthermore the great variability in distribution of acceleration pulse amplitude between different ground motion records produces great variability in predicted slope displacements; even ground motions with similar amplitudes, frequency contents and durations can produce significantly different predicted slope displacements. It is therefore necessary to carry ot the analysis for a large number of relevant ground motion records and to apply statistical techniques to the results.
- The results of dynamic FEM analyses depend on the costitutive model used; as far as the input ground motion is concerned, the same considerations apply as detailed above for Newmark type analyses.

MITIGATION THROUGH REDUCTION OF HAZARD

2 MODIFYING THE SLOPE GEOMETRY/MASS DISTRIBUTION

2.0 GENERAL

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2 MODIFYING THE SLOPE GEOMETRY/MASS DISTRIBUTION

2.0 GENERAL

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MITIGATION THROUGH REDUCTION OF HAZARD

2 MODIFYING THE SLOPE GEOMETRY/MASS DISTRIBUTION

2.1 REMOVAL OF (ACTUAL OR POTENTIALLY) UNSTABLE SOIL/ROCK MASS

Description

In some situations, complete removal of the actually or potentially unstable mass can be an effective and economic form of mitigation, removing the potential hazard at source. Probably the most high profile example of the application of this technique is the construction and maintenance of the Panama Canal (Duncan, 2008). Generally, however, it is only practical on small slumps or small rotational slides. Large scale excavation of larger landslide areas is usually not recommended for several reasons (Highland and Bobrowsky, 2008):

- Excavation is not always effective. For large planar failures, excavation may not cause movements to stop and may allow the landslide to expand.
- Excavation may actually destibilize the ground further upslope by undercutting, which weakens the slope, even to the point of triggering a larger slide than is being mitigated.
- In certain soil profiles, where there are several actual or potential failure surfaces at different depths, excavating down to the top failure surface might trigger sliding on deeper failure surfaces.
- Excavation may interfere with surface runoff and water courses; unless this is properly addressed, it may cause backdrops and areas of temporary or permanent stagnant water with resulting changes to infiltration and the groundwater regime of the slope, or favouring erosion in areas previously protected by the slide mass.

Complete removal of the landslide body is only effective at mitigating the hazard of further movement if the slope can be reprofiled at a lower inclination compared to the original, failed, slope and/or additional hazard mitigation measures are implemented. If this is not carried out, removing the landslide debris is equivalent to accelerated erosion at the toe of the slope where this was a cause of the original landslide, recreating the conditions for further sliding to take place.

Excavation may alter drainage patterns, with potentially detrimental effects on the stability of the area; care should be paid to diverting surface water flows away from the excavated areas and to ensure that reprofiling does not create conditions for stagnant water to accumulate in low lying areas. Similarly, it is necessary to ensure that the materials exposed by the removal of the landslide body are not susceptible to or are adequately protected from rapid weathering which could cause renewed landsliding. To facilitate construction and maintenance of drainage and surgface protection works, excavated surfaces are typically shaped to form a number of benches, typically at 6 to 10 m vertical interval.

The equipment and methods of excavation will need to be selected to suit the nature of the material to be excavated and local conditions in general. Even when the parent, undisturbed material is rock, landsliding may have broken up the mass sufficiently for it to be excavated by conventional equipment. However, it is not rare for the landslide debris to retain sufficient remnants of the original structure and consistency of the parent material that excavation and removal of the landslide debris requires specialist equipment, such as hydraulic hammers or even explosives. In these situations, careful consideration will be given to the need to minimize vibrations, if there is a risk of these triggering further movement.

Design

In all cases a careful review of ground, groundwater and drainage conditions needs to be undertaken before any excavation is carried out. When considering complete removal of the landslide body, it is necessary to evaluate the stability of the slope in the proposed final configuration, with particular attention to the stability of the slope above the excavated area. The principles and methods of analysis are decribed in the general fact-sheet 2.0 on "Hazard reduction by modifying the slope geometry or mass distribution".

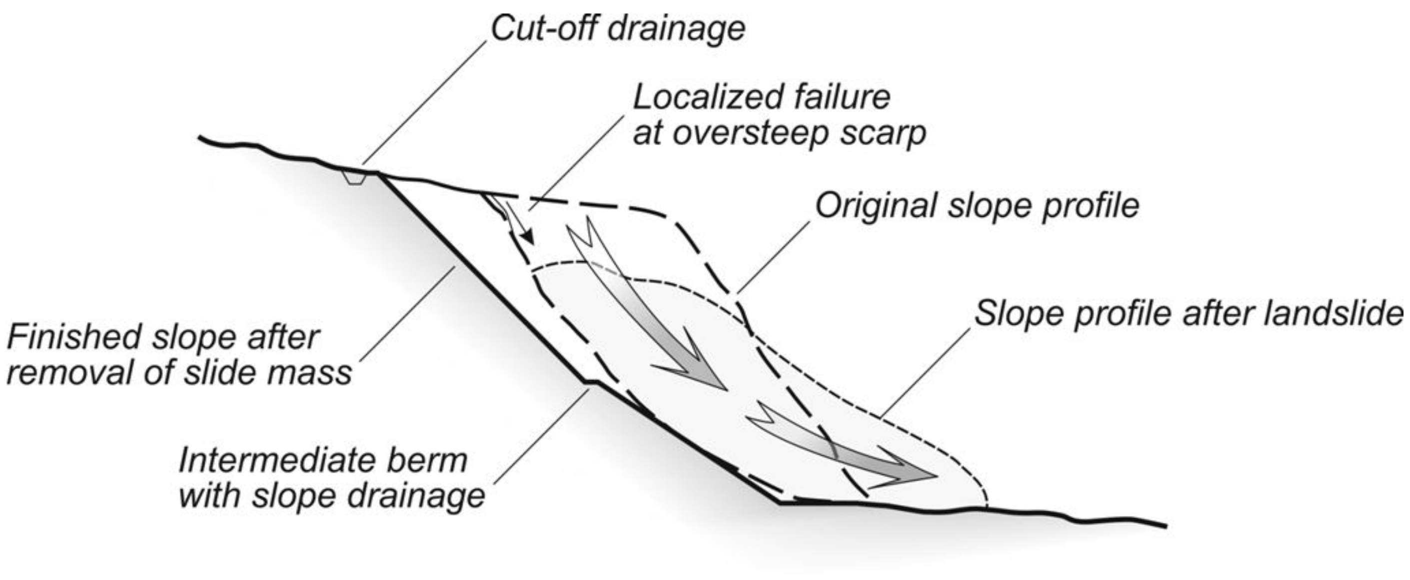
The design should consider the method and sequence of excavation, to ensure stability at all times, especially when excavating active landslides; typically, excavation should proceed from the top of the slope downwards, rather than from the toe, to ensure that the work is carried out safely.

The design should also consider the final disposal of the excavated material, which can be a serious problem in some cases. Uncontrolled tipping of the material downslope of the excavated area, as often practiced in emergency rehabilitation of rural roads in mountainous terrain, should be avoided since it can damage the existing vegetation and it can create a serious hazard of further sliding downhill.

Special care needs to be paid if the landslide mass is suspected to contain contaminated materials, for whatever reason, since this may require special provision with respect to ensuring the safety of both workers and the public and with respect to arrangements for the disposal of arisings.

Finally, complete removal of the actual or potential landslide may have a significant visual impact on the landscape, which needs to be considered and weighed against the cost of possible alternatives.

Figure 1: Schematic section



Picture 1: Landslide debris tipped over the side in emergency road rehabilitation. *This practice should be avoided as far as possible* (photograph from http://bengalnewz.blogspot.com/2010/08/maoist-frontal-organisation-pcpa-may.html)

MITIGATION THROUGH REDUCTION OF HAZARD			
2 MODIFYING THE SLOPE GEOMETRY/MASS DISTRIBUTION			
2.1 REMOVAL OF (ACTUAL OR POTENTIALLY) UNSTABLE SOIL/ROCK MASS			
APPLICABILITY			
Class	**Descriptor**	**Rating**	**Notes**
Type of movement (Cruden & Varnes, 1996)	Falls	4	Most applicable to slides, although might cause further sliding in certain conditions. Applicable in principle also to rock slopes subject to falls or toppling. The complete removal of source material for potential flows may be considered in special circumstances but it is unlikely to be applicable in practice.
	Topples	4	
	Slides	6	
	Spreads	0	
	Flows	2	
Material	Earth	8	Mainly applicable to landsliding involving earth and debris. Applicability in rock limited by difficulty of excavation.
	Debris	8	
	Rock	4	
Depth of movement	Superficial (< 0.5 m)	10	Typically applicable to relatively small and/or shallow landslides. The implications of large scale excavation and disposal typically make this technique impractical for deep and very deep slides.
	Shallow (0.5 to 3 m)	6	
	Medium (3 to 8 m)	4	
	Deep (8 to 15 m)	2	
	Very deep (> 15 m)	0	
Rate of movement (Varnes, 1978)	Moderately to fast	2	Can be carried out without special difficulty when the rate of movement is slow (5 cm/day) or less; in certain circumstances and with due care, it is possible to excavate slides moving moderately fast (up to a few metres per day), especially if it is possible to place the equipment on stable ground.
	Slow	6	
	Very slow	8	
	Extremely slow	8	
Groundwater	Artesian	2	High or artesian groundwater conditions pose special problems both to the excavation and to the stability of the slope after removal of the landslide mass, limiting the applicability of this technique when these conditions occur.
	High	4	
	Low	8	
	Absent	8	
Surface water	Rain	6	Surface flows must be diverted to prevent them from reaching the excavated area.
	Snowmelt	6	
	Localized	0	
	Stream	0	
	Torrent	0	
	River	0	
Maturity		10	Simple technique. Potential benefits and limits of applicability are well established.
Reliability		8	The reliability of the technique as a mitigation measure depends on the reliability of the evaluation of the stability of the treated slope.
Implementation		8	Easily implemented with widely available equipment. Possible difficulties with excavation in rock and with the disposal of arisings.
Typical Cost		6	Moderate, provided the work does not involve contaminated material.

Note
Ratings are given on a scale of 1 to 10; the higher the grade, the most suitable is the specific method under consideration to use in landslides of the given characteristics, evaluated individually. Overall suitability to specific case under consideration may be obtained by a weighted average of these ratings, with user defined weights. Zero rating means "not applicable"

MITIGATION THROUGH REDUCTION OF HAZARD	
2 MODIFYING THE SLOPE GEOMETRY/MASS DISTRIBUTION	
2.1 REMOVAL OF (ACTUAL OR POTENTIALLY) UNSTABLE SOIL/ROCK MASS	
References: Duncan J.M. (2008) “Managing landslides in the Panama Canal”. Proc. 33d International Geological Congress, Oslo. Highland L.M., Bobrowsky P. (2008). “The landslide handbook – a guide to understanding landslides”. Circular 1325, U.S. Geological Survey Hutchinson J.N. (1977). “Assessment of the effectiveness of corrective measures in relation to geological conditions and types of slope movement”. General Report. Bulletin of Engineering Geology and the Environment, 16, n°1, 131-155. Turner A.K., Schuster R.L. eds. (1996). “Landslides: Investigation and Mitigation”. Special Report 247, Transportation Research Board, Washington.	

MITIGATION THROUGH REDUCTION OF HAZARD

2 MODIFYING THE SLOPE GEOMETRY/MASS DISTRIBUTION

2.2 REMOVAL OF LOOSE OR POTENTIALLY UNSTABLE BLOCKS/BOULDERS (TRIMMING AND SCALING)

Description

Loose or potentially unstable vegetation, blocks and localized bulges or overhangs remaining on a rock slope as a result of previous falls or excavations, progressive loosening and weathering of discontinuities as a result of freeze-thaw cycles, growing tree roots, weathering and/or washing out of clayey infill in rock discontinuities can be removed to sound rock by a variety of means and techniques, collectively referred to as "trimming" and "scaling". Larger scale reprofiling of rock slopes falls within the scope of sheet 2.1 – "Removal of (actually or potentially) unstable soil/rock mass". In particular, "trimming" refers mainly to the removal of potentially unstable overhangs, bulges and other geometric anomalies protruding above the general lay of the slope, while "scaling" refers to the removal of individual blocks or boulders which may be or readily become detached from the slope, even if they do not represent a geometric anomaly.

Trimming and scaling may be carried out by a variety of methods and techniques, depending on the size of anomaly to be removed and, even more importantly, access conditions. While scaling can be carried out, to some extent, by conventional hand held tools, such as pry bars, shovels, etc., this may need to be supplemented by controlled blasting or other significant mechanical assistance, especially when trimming involves the removal of blocks which are not yet detached.

Where small scale blasting is used, blast mats may need to be used to prevent flying debris, since typically there will be insufficient overburden to provide confinement. To minimize the risk of blasting causing uncontrolled fracturing of the rock, requiring additional trimming and scaling, cotrolled blasting is typically carried out by drilling one or more series of closely spaced (typically at 10 to 12 times the diameter), parallel holes along the intended breakline to evenly distribute the explosive on the face. Holes drilled by hand-held equipment are normally up to 40mm diameter and up to 3 m in length. Typically, low-velocity explosive is used, with a decoupling ratio (the ratio between the the hole diameter and that of the explosive) of about 2 to limit the pressure on the side of the hole to limit uncontrolled fracturing, stemming the holes to minimize venting and detonating each hole on a single delay. Where the thickness of material to be removed is significant, multiple breaking lines are detonated in sequence, starting from that nearest the free face, to the final line.

Blasting is often precluded by regulations in or near urban areas. Blanket bans on blasting may be in force in some countries or it may be so cumbersome to obtain permission for and to actually carry out blasting that to all effects this option is not available. In this case, alternative methods of demolition may be considered, depending on circumstances, including:

- hydraulic hammers (rock breakers), either hand held or mounted on the boom of an excavator;
- hydraulic rock splitters, which are inserted in a line of drilled holes and expanded hydraulically to create or open cracks;
- expansive grouts (soundless chemical demolition agents), which expand slowly as a result of chemical reactions.

Both trimming and scaling can be highly dangerous and need to be carried out by specialist personnel operating under a strict safety regime. Typically the work is carried out proceeding from the po of the slope downwards, so that the workers are not unnecessarily exposed to the hazard of material falling from above and to avoid that the debris from the operation accumulates on previously completed portions of the slope.

Workers and equipment are typically suspended from ropes anchored in a safe area above the slope. On smaller slopes, access can be provided by self elevating platforms, with heavier equipment suspended from cranes, but this arrangement tends to be cumbersome and does not afford workers the same freedom of movement in case of need.

Since the debris from these operations will fall to the base of the slope, access to this area must be restricted during this type of work and the exclusion zone must extend sufficiently to cover for all possible run-out trajectories. Vulnerable structures within the exclusion zone may need to be temporarily protected.

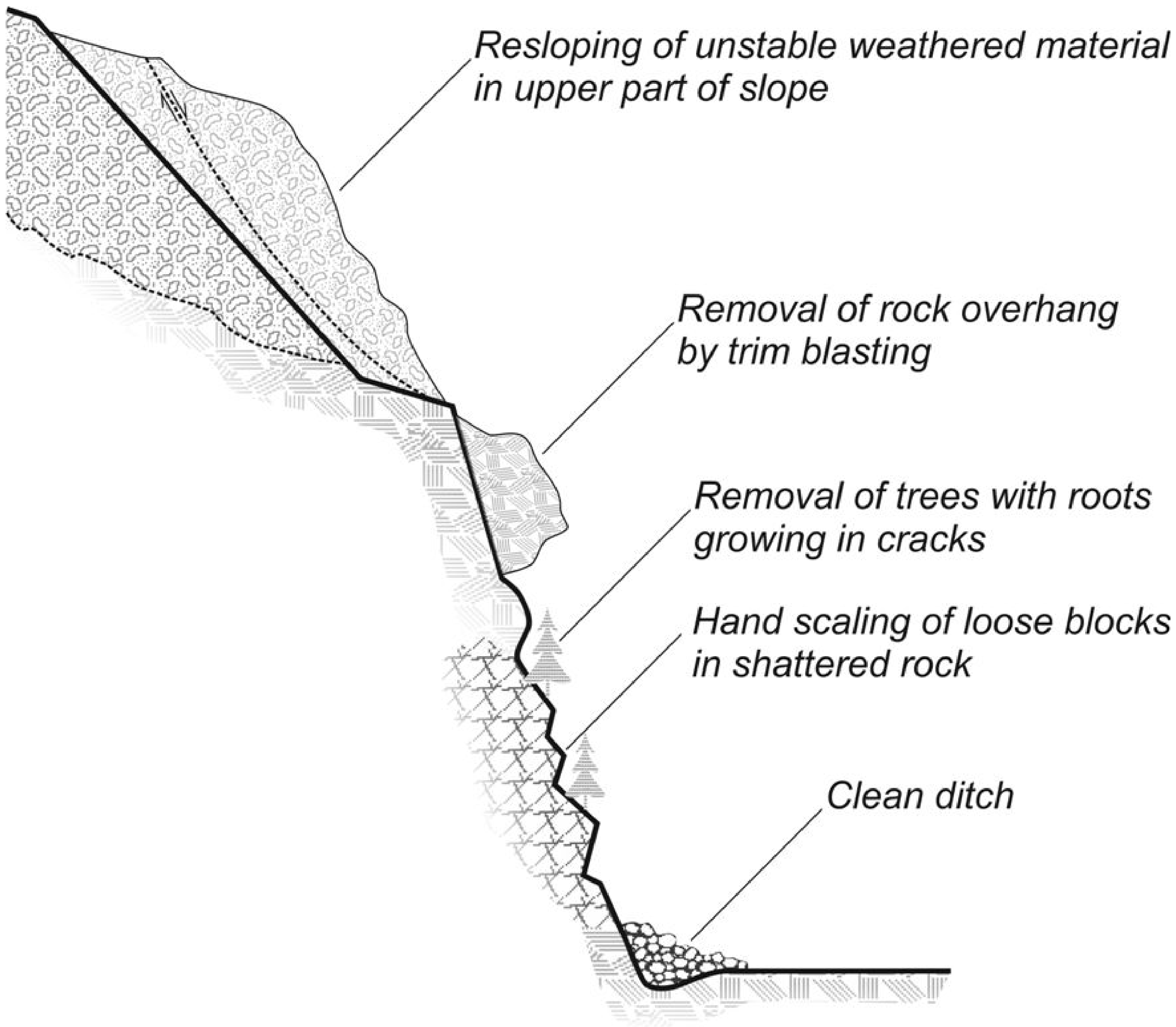
Trimming and scaling may need to be repeated at regular intervals, especially if the rock is susceptible to rapid weathering, for example in mountain areas subjected to repeated freeze-thaw cycles, or where the rock face is overlain by debris.

Design

The design of scaling and trimming does not typically involve calculation. Rather, the design involves the identification and mapping of the main unstable blocks, bulges and overhangs that need to be removed, delegating to some extent to the workers on the face the task to determine whether a specific block needs to be removed, preferably to pre-defined criteria.

In relation to the need to define a safety exclusion zone and to protect vulnerable structures from falling debris, computer programs may be used to simulate the trajectories of falling rocks as they bouce down the slope (Piteau, 1980; Wu, 1984; Descoeudress and Zimmerman, 1987; Spang, 1987; Hungr and evans, 1988; Pfeiffer and Bowen, 1989, Pfeiffer et al., 1990). These programmes require information on the geometry and roughness of the rock face, the attenuation characteristics of the materials and details of the size and shape of the blocks. The statistical analysis of the results of a large number of simulations may be used to estimate the optimum position and dimensions of ditches and the height and capacity of fences and barriers.

Figure 1: Schematic (source: Wyllie and Norrish, 1996)

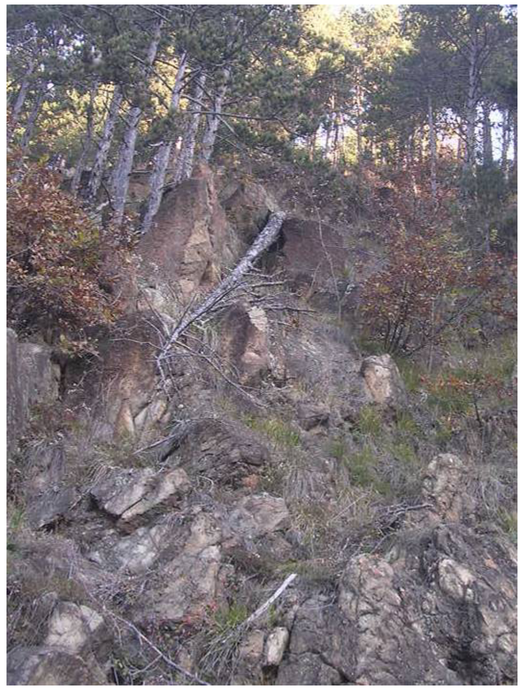


MITIGATION THROUGH REDUCTION OF HAZARD

2 MODIFYING THE SLOPE GEOMETRY/MASS DISTRIBUTION

2.2 REMOVAL OF LOOSE OR POTENTIALLY UNSTABLE BLOCKS/BOULDERS (TRIMMING AND SCALING)

Pictures 1 and 2: Typical situation requiring scaling, at different scales (source: SGI-MI project files)



Pictures 4: Typical scaling and trimming work with rope access (source: http://pacificblasting.com/stabilization.html)

Picture 3: Typical situation requiring trimming (source: SGI-MI project files)

Picture 5: Scaling and trimming work by long reach equipment (source: Highland and Bobrowski, 2008)

MITIGATION THROUGH REDUCTION OF HAZARD

2 MODIFYING THE SLOPE GEOMETRY/MASS DISTRIBUTION

2.2 REMOVAL OF LOOSE OR POTENTIALLY UNSTABLE BLOCKS/BOULDERS (SCALING)

APPLICABILITY

Class	Descriptor	Rating	Notes
Type of movement (Cruden & Varnes, 1996)	Falls	8	Only suitable to prevent/anticipate falls and, to a lesser extent topples, of individual blocks.
	Topples	6	
	Slides	0	
	Spreads	0	
	Flows	0	
Material	Earth	2	Applicable to rock slopes and, to a much lesser extent, to cemented soils.
	Debris	0	
	Rock	8	
Depth of movement	Superficial (< 0.5 m)	8	Applicable to superficial or very shallow movement. Large scale reprofiling to be considered separately.
	Shallow (0.5 to 3 m)	2	
	Medium (3 to 8 m)	0	
	Deep (8 to 15 m)	0	
	Very deep (> 15 m)	0	
Rate of movement (Varnes, 1978)	Moderately to fast	0	Rock face must be stable; conditions should not be conductive to falls occurring whilst the work is being carried out.
	Slow	0	
	Very slow	8	
	Extremely slow	8	
Groundwater	Artesian	0	Generally most suitable in dry conditions or minor seepage from the face; in other conditions it needs to be supllemented and preceeded by drainage.
	High	2	
	Low	8	
	Absent	10	
Surface water	Rain	6	Suitable to reduce hazard associated with rainfall, snowmelt and freeze-thaw cycles and intermittent localized flows over the face.
	Snowmelt	8	
	Localized	4	
	Stream	0	
	Torrent	0	
	River	0	
Maturity		8	Widespread experience.
Reliability		8	High, provided parent material not susceptible to rapid weathering, in which case it may need to be repeated on a regular basis.
Implementation		4	Difficult and hzardous.
Typical Cost		8	Relatively low.

Note
Ratings are given on a scale of 1 to 10; the higher the grade, the most suitable is the specific method under consideration to use in landslides of the given characteristics, evaluated individually. Overall suitability to specific case under consideration may be obtained by a weighted average of these ratings, with user defined weights. Zero rating means "not applicable"

MITIGATION THROUGH REDUCTION OF HAZARD

2 MODIFYING THE SLOPE GEOMETRY/MASS DISTRIBUTION

2.2 REMOVAL OF LOOSE OR POTENTIALLY UNSTABLE BLOCKS/BOULDERS (SCALING)

References:

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MITIGATION THROUGH REDUCTION OF HAZARD

2 MODIFYING THE SLOPE GEOMETRY/MASS DISTRIBUTION

2.3 REMOVAL OF MATERIAL FROM DRIVING AREA

Description

The removal of material from the driving area (or more in general, regrading or flattening slope angle) operates by reducing the driving forces, thereby improving overall slope stability.

This method is most suitable in cases where the instability mechanism occurs as a rotational or pseudo-rotational slide, e.g. where the displaced mass moves as a relatively coherent mass along a spoon-shaped (curved upward) failure surface with little internal deformation. It is generally ineffective on translational slides on long, uniform planar slopes, or on flow-type landslides.

Generally it is most practical on small slumps or small rotational failures, but several examples exist where this technique has been applied successfully on large landslides where conditions allowed large scale earthmoving to be carried out.

It should always be kept in mind that the resisting forces are also reduced, especially in the long term, as a result of the reduction in normal stress on the failure surface. It is therefore necessary to locate the excavation in such manner that the reduction in driving forces exceeds the reduction in resisting forces. The neutral line concept, described in fact sheet 2.0 on “mitigation by modifying the slope geometry / mass distribution; general aspects” can be used for a preliminary evaluation of the relative merits of the proposed excavation.

The main limitations of the technique relate to the following issues:

- Excavation may actually destabilize the ground farther up-slope by ubdercutting ;
- Excavation increases safety factor by only a limited amount, which tends to decrease with time in low permeability saturated soils; satisfactory solutions may involve significant modification of the landscape (see for example Chatwin et al.,1994);
- Excavation results in large volumes of material to be disposed of off-site in a controlled manner, with attendant difficulties;
- Excavation may interfere with existing structures and services; This is potentially significant when considering this type of mitigation for “potential” landslides, while on actual landslides the residual value of existing structures and facilities can be very low;
- Excavation impacts on the upper part of the slope, with the greatest potential visual impact on the landscape
- Excavation of active landslides requires special care to ensure the safety of workers; in particular, it is necessary to assess the possibility of sudden accelerations and to have in place well drilled evacuation plans.

All excavation in the upper part of a landslide must be accompanied by drainage works to redirect surface water away from infiltrating the landslide body. Typically, surface protection to newly excavated surfaces is also necessary to limit erosion and/or weathering. To facilitate construction and maintenance of drainage and surgface protection works, excavated surfaces are typically shaped to form a number of benches, typically at 6 to 10 m vertical interval.

Examples of large landslides stabilized by this technique are shown in Figures 1 and 2.

Figures 3 and 4 show the remedial works carried out at the Settebagni motorway cutting, just North of Rome, where major deep seated sliding occurred approximately 20 - 25 years after construction due to a thick plio-pleistocene clay layer daylighting in the cutting below a thick cover of otherwise stable tuffs and pyroclastic cinders (Pictures 1 and 2). The extent of the clay outcrop in the cutting is shown indicatively by the hard facing installed at the time of construction to safeguard fron erosion and shallow instability. As shown in the figures, reprofiling formed an essential part of stabilization works and extended for the full portion of the cutting potentially affected by future sliding, beyond the limits of the 1992 slide (SGI-MI project files).

Design

For general considerations on the geotechnical design of mitigation by removal of material from the driving are, reference shall be made to the general fact sheet 2.0 on hazard mitigation by changes in slope geometry and/or mass distribution.

Figuere 1: Cameo Slide, Colorado River Valley – Stabilization by partial removal of material in driving area (Volume B). Removal of volume A was considered and found ineffective (source Peck and Ireland, 1953; Baker and Marshall, 1958)

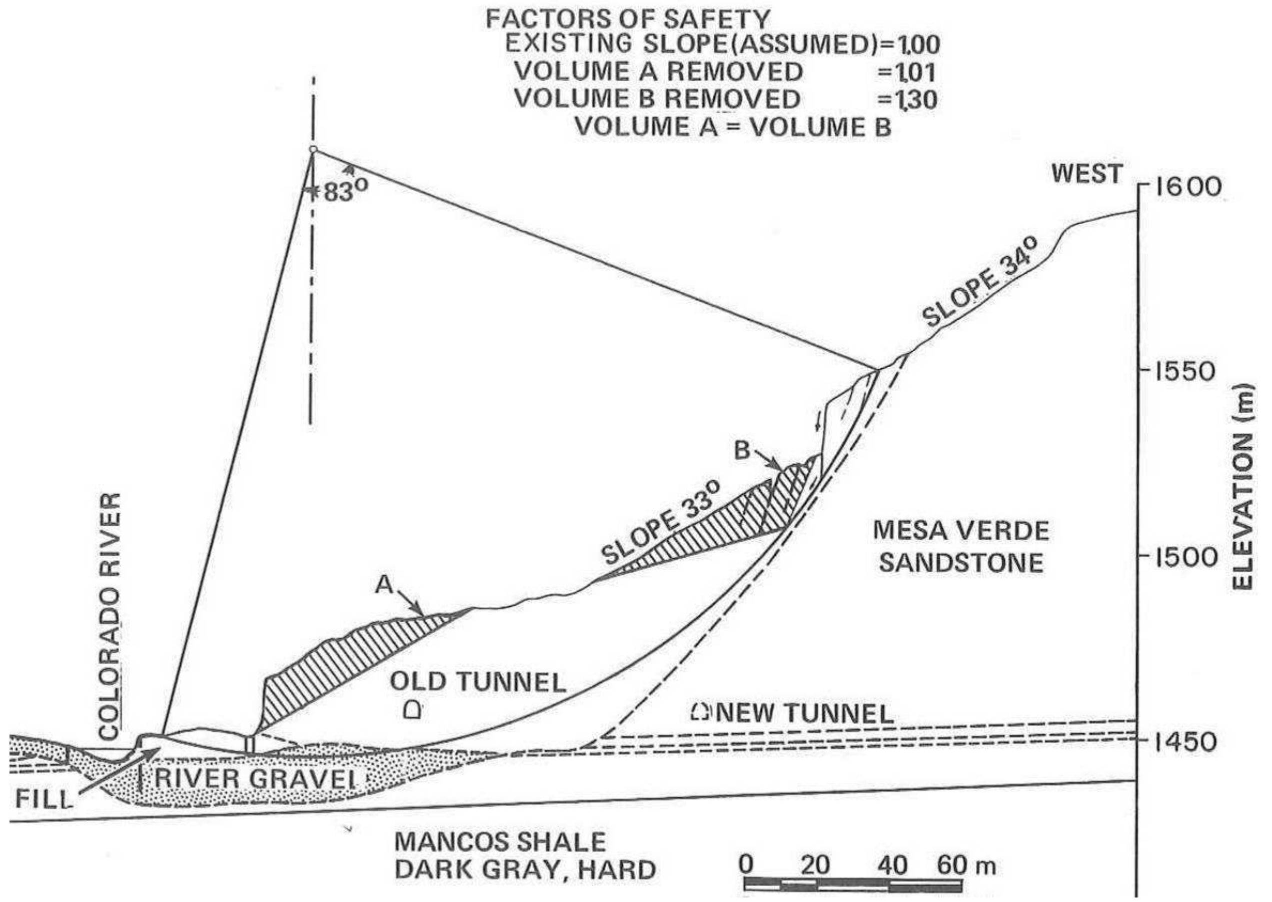
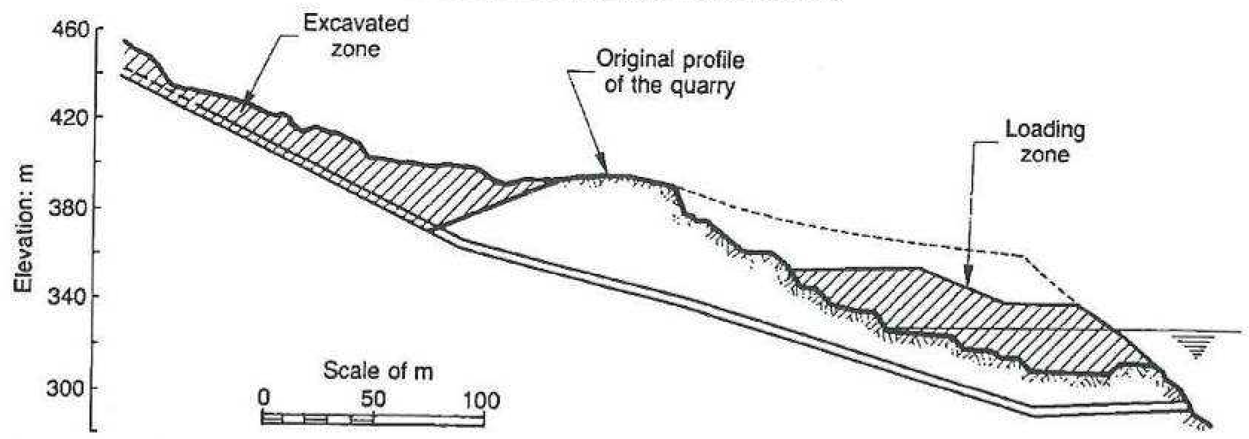


Figure 2: Cortes de Pallas Landslide, Spain – Stabilization by excavation in driving area (source Alonso et al., 1993)



MITIGATION THROUGH REDUCTION OF HAZARD

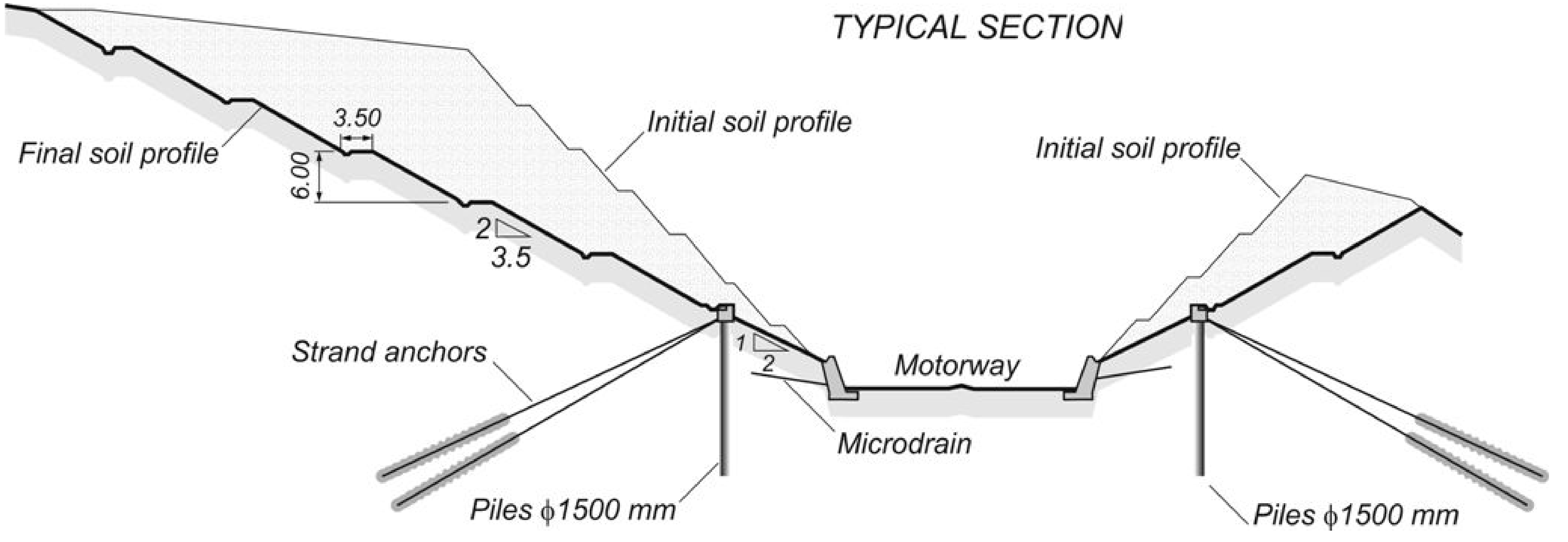
2 MODIFYING THE SLOPE GEOMETRY/MASS DISTRIBUTION

2.3 REMOVAL OF MATERIAL FROM DRIVING AREA

Picture 1 Settebagni slide – 1992 after emergency temporary remedial works (SGI-MI project files)

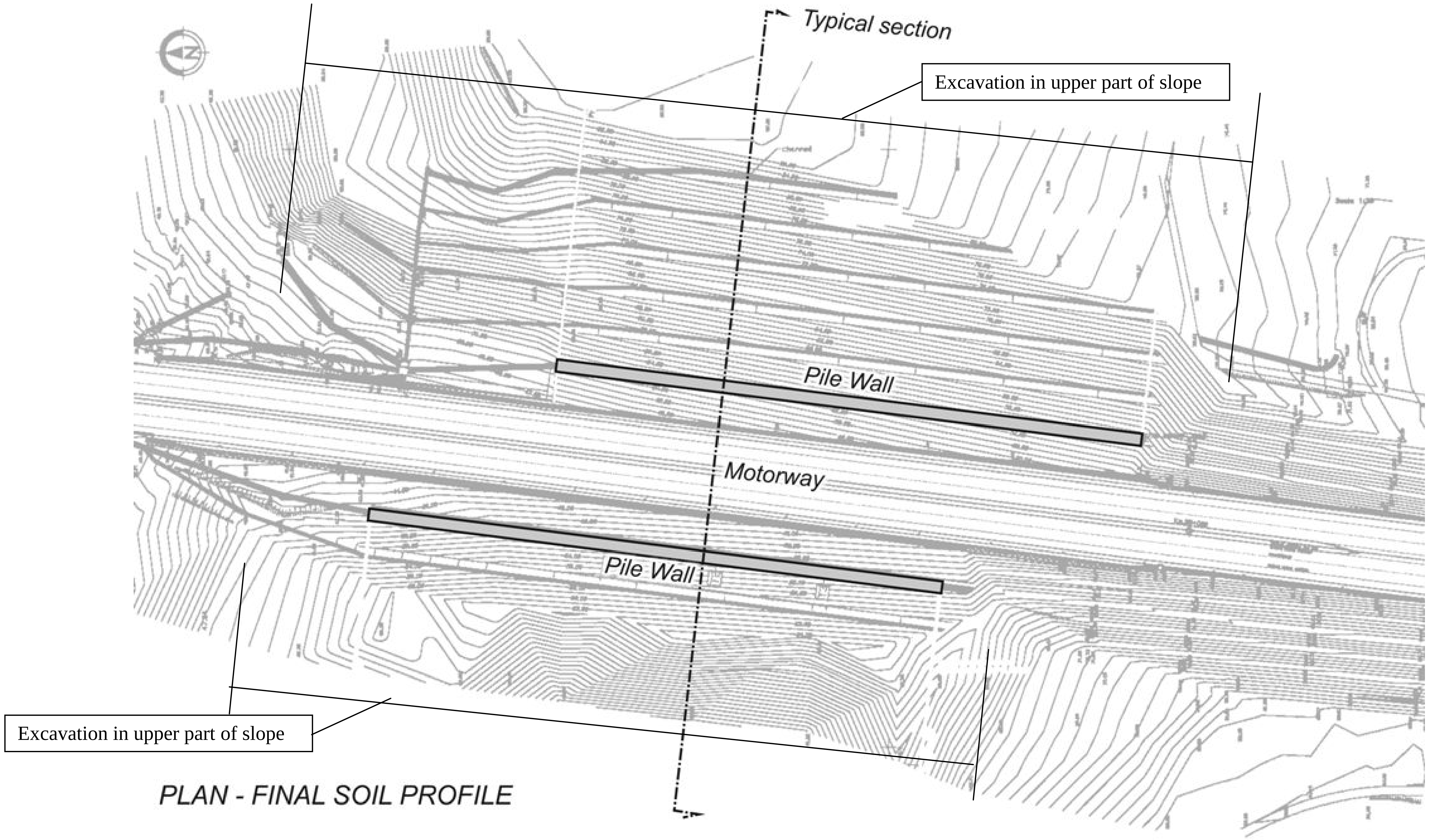
Picture 2 Settebagni slide – 1992 after emergency temporary remedial works (SGI-MI project files)

Figure 3: Remedial works for Settebagni slide included major reprofiling from the original 1960's cut slope profile to reduce driving forces (SGI-MI project files)



MITIGATION THROUGH REDUCTION OF HAZARD
2 MODIFYING THE SLOPE GEOMETRY/MASS DISTRIBUTION
2.3 REMOVAL OF MATERIAL FROM DRIVING AREA

Figure 4: Remedial works for Settebagni slide included major reprofiling from the original 1960's cut slope profile to reduce driving forces; note original grading in southern portion of cut (SGI-MI project files)



MITIGATION THROUGH REDUCTION OF HAZARD			
2 MODIFYING THE SLOPE GEOMETRY/MASS DISTRIBUTION			
2.3 REMOVAL OF MATERIAL FROM DRIVING AREA			
APPLICABILITY			
Class	**Descriptor**	**Rating**	**Notes**
Type of movement (Cruden & Varnes, 1996)	Falls	0	Most suited to rotational or pseudo-rotational slides; may be useful to reduce toppling hazard in certain conditions.
	Topples	2	
	Slides	8	
	Spreads	0	
	Flows	0	
Material	Earth	8	Mainly applicable to landsliding involving earth and debris. Applicability in rock limited by difficulty of excavation.
	Debris	8	
	Rock	4	
Depth of movement	Superficial (< 0.5 m)	8	Typically applicable to relatively small and/or shallow landslides. The implications of large scale excavation and disposal typically make this technique impractical for deep and very deep slides. On the other hand, it may be the only suitable technique in very large landslides, besides drainage.
	Shallow (0.5 to 3 m)	8	
	Medium (3 to 8 m)	8	
	Deep (8 to 15 m)	6	
	Very deep (> 15 m)	6	
Rate of movement (Varnes, 1978)	Moderately to fast	2	Can be carried out without special difficulty when the rate of movement is slow (5 cm/day) or less; in certain circumstances and with due care, it is possible to excavate slides moving moderately fast (up to a few metres per day), especially if it is possible to place the equipment on stable ground.
	Slow	8	
	Very slow	8	
	Extremely slow	8	
Groundwater	Artesian	4	High or artesian groundwater conditions pose special problems both to the excavation and to the stability of the slope after removal of the landslide mass, limiting the applicability of this technique when these conditions occur.
	High	6	
	Low	8	
	Absent	8	
Surface water	Rain	6	Surface flows must be diverted to prevent them from reaching the excavated area, infiltrating the portion of the landslide mass left in place.
	Snowmelt	6	
	Localized	4	
	Stream	2	
	Torrent	0	
	River	0	
Maturity		8	Simple technique. Potential benefits and limits of applicability are well established.
Reliability		6	The reliability of the technique as a mitigation measure depends on the reliability of the evaluation of the stability of the treated slope.
Implementation		8	Easily implemented with widely available equipment. Possible difficulties with excavation in rock and with the disposal of arisings.
Typical Cost		8	Moderate, provided the work does not involve contaminated material.

Note
Ratings are given on a scale of 1 to 10; the higher the grade, the most suitable is the specific method under consideration to use in landslides of the given characteristics, evaluated individually. Overall suitability to specific case under consideration may be obtained by a weighted average of these ratings, with user defined weights. Zero rating means "not applicable"

MITIGATION THROUGH REDUCTION OF HAZARD	
2 MODIFYING THE SLOPE GEOMETRY/MASS DISTRIBUTION	
2.3 REMOVAL OF MATERIAL FROM DRIVING AREA	
References: Alonso E.E, Gens A., Lloret A. (1993). "The landslide of Cortes de Pallas, Spain". Geotechnique, Vol. 43 (4), 507-521 Baker R.F., Marshall H.C., (1958). "Control and correction". In: Special Report 29: Landslides and Engineering Practice, E.B. Eckel (eds.), HRB, National Research Council, Washington D.C., 150-188. Chatwin S.C., Hiwes D.E., Schwab J.W., Swanston D.N. (1994). "A guide for management of landslide-prone terrain in the Pacific Northwest". Second Edition, Ministry of Forest. http://www.for.gov.bc.ca/hfd/pubs/docs/Lmh/Lmh18.htm . Peck R.B., Ireland H.O. (1953). "Investigation of stability problems". Proc. American railway Engineering Association. Vol. 54, 1116-1128.	

MITIGATION THROUGH REDUCTION OF HAZARD

2 MODIFYING THE SLOPE GEOMETRY/MASS DISTRIBUTION

2.4 SUBSTITUTION OF MATERIAL IN DRIVING AREA WITH LIGHTWEIGTH FILL

Description

This mitigation technique consists in excavating the material in the driving area and replacing it with a lightweight backfill material (Figure 1).

Lightweight fill is also used to minimize the extent and cost of other mitigation measures by minimizing the adverse effect of construction, for example where alignment constraints may dictate that fills for a new highway be placed in a potentially destabilizing position across an actual or potential landslide.

A wide variety of lightweight materials have been proposed and used in this context, depending on local availability and practice and reguloatory constraints, including naturally (geological) lightweight materials such as pumice or shells, manufactured materials, such as expanded clay, polystyrene slabs, cellular concrete, and waste materials or byproducts, such as soil mixed with shredded tyres (‘pneusol’), pulverized fly ash, slag, woodchips or logging slash. Clearly, manufactured materials are typically more expensive and synthetic material may have limited durability, but they afford greater reliability in terms of homogeneity of results; the use of waste materials or byproducts may also be subject to environmental constraints and concerns about possible long term pollution.

This technique operates on the principle of reducing the driving forces more than the resisting forces by altering the mass or load distribution on the slope, in the same manner and subject to similar considerations and limitations as described in fact-sheet 2.3 on “Removal of material from the driving area”. It is most suitable in cases where the instability mechanism occurs as a rotational or pseudo-rotational slide, e.g. where the displaced mass moves as a relatively coherent mass along a spoon-shaped (curved upward) failure surface with little internal deformation, while it is generally ineffective on translational slides on long, uniform planar slopes, or on flow-type landslides.

It should always be kept in mind that the resisting forces are also reduced, especially in the long term, as a result of the reduction in normal stress on the failure surface. It is therefore necessary to locate the excavation in such manner that the reduction in driving forces exceeds the reduction in resisting forces. The neutral line concept, described in fact sheet 2.0 on “mitigation by modifying the slope geometry / mass distribution; general aspects” can be used for a preliminary evaluation of the relative merits of the proposed excavation.

Generally it is most practical where it is necessary to remediate or prevent small slumps or small rotational failures, while at the same time maintaining a specific function. It is generally impractical and not necessary to carry out large scale substitution as would be necessary on large landslides.

Compared to the simple removal of the landslide mass in whole (2.1) or in part (2.3), substitution affords long term surface protection to the excavated surface. However, the permeability of the lightweight fill is typically much higher than that of the original soil and special care must be paid to drainage, both at the surface and at the interface with the natural soil.

The main limitations of the technique relate to the following issues:

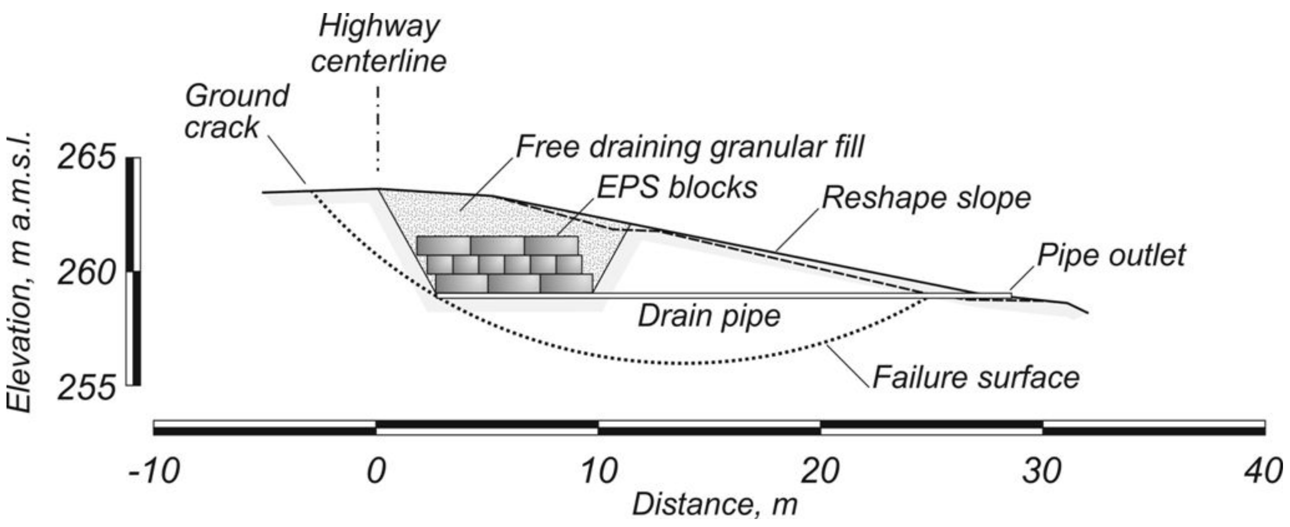
- Excavation and replacement with lightweight fill may actually destabilize the ground farther up-slope by ubdercutting;
- Excavation and replacement with lightweight fill increases safety factor by only a limited amount, which tends to decrease with time in low permeability saturated soils;
- Excavation results in large volumes of material to be disposed of off-site in a controlled manner, with attendant difficulties;
- Excavation may interfere with existing structures and services; This is potentially significant when considering this type of mitigation for “potential” landslides, while on actual landslides the residual value of existing structures and facilities can be very low;
- Work on active landslides requires special care to ensure the safety of workers; in particular, it is necessary to assess the possibility of sudden accelerations and to have in place well drilled evacuation plans.
- Vibration necessary to compact certain lightweight fills may be detrimental to slope stability.

Design

For general considerations on the geotechnical design of mitigation by removal of material from the driving are, reference shall be made to the general fact sheet 2.0 on hazard mitigation by changes in slope geometry and/or mass distribution.

For the mechanical characteristics of manufactured materials, reference may be made to published guidelines (see for example Stark et al., 2004 on geofoam; Di Prisco, 2007 on expanded clay).

Figure 1: Schematic section (source: Geofoam Research Centre, Syracuse University, http://geofoam.syr.edu/GRC_bayfd.asp)



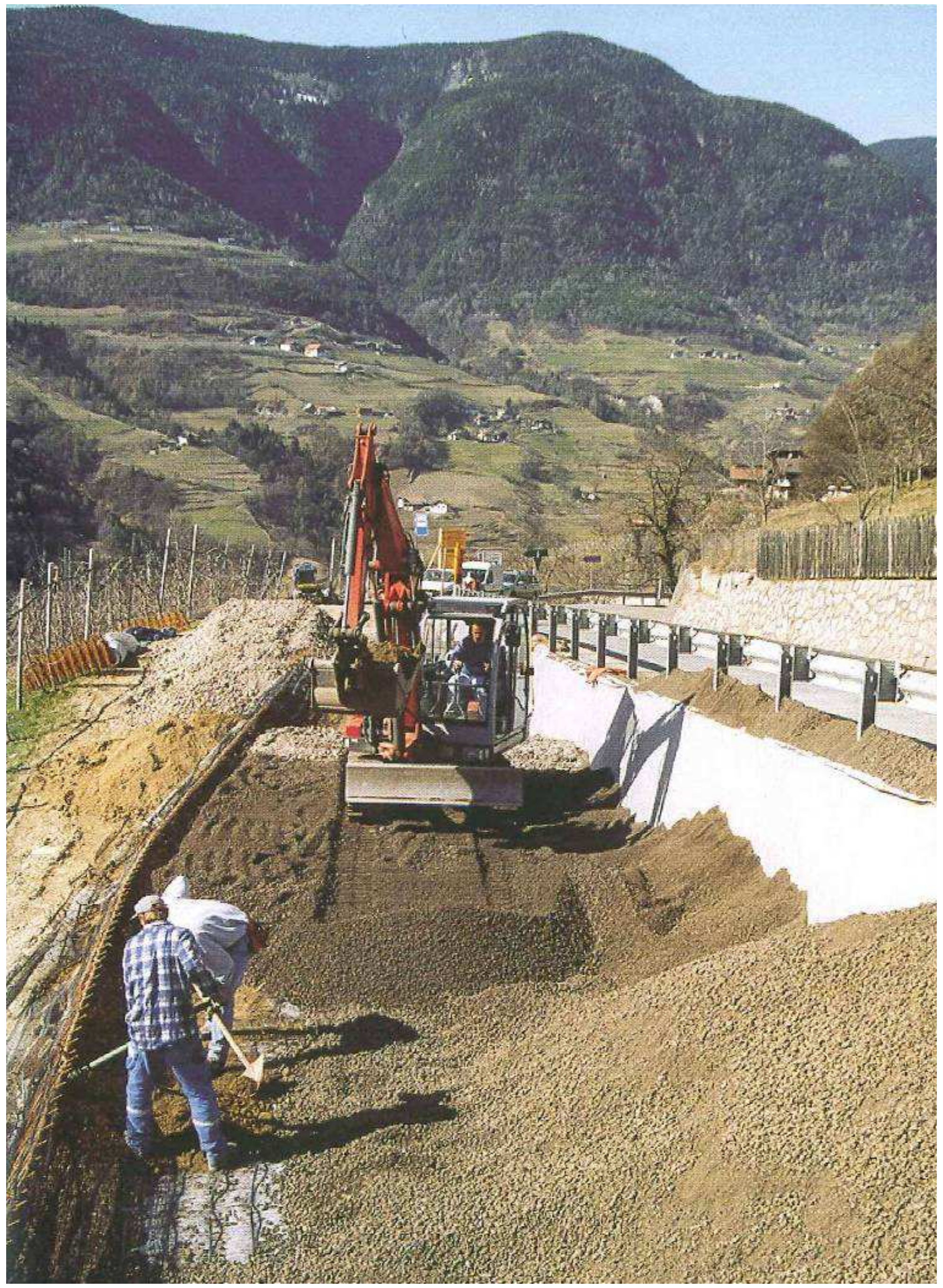
Picture 1: Installation of geofoam mats for lanslide remediation (source: Geofoam Research Centre, Syracuse University, http://geofoam.syr.edu/GRC_bayfd.asp)

MITIGATION THROUGH REDUCTION OF HAZARD

2 MODIFYING THE SLOPE GEOMETRY/MASS DISTRIBUTION

2.4 SUBSTITUTION OF MATERIAL IN DRIVING AREA WITH LIGHTWEIGTH FILL

Picture 2: Installation of expanded clay for lanslide remediation (source: Di Prisco, 2007)



Picture 3: Installation of expanded clay for lanslide remediation (source: Di Prisco, 2007)

Picture 4: Use of shredde tyres for lanslide remediation (source: Dubreucq T. and Pezas N., 2009)

MITIGATION THROUGH REDUCTION OF HAZARD			
2 MODIFYING THE SLOPE GEOMETRY/MASS DISTRIBUTION			
2.4 SUBSTITUTION OF MATERIAL IN DRIVING AREA WITH LIGHTWEIGTH FILL			
APPLICABILITY			
Class	**Descriptor**	**Rating**	**Notes**
Type of movement (Cruden & Varnes, 1996)	Falls	0	Only suited to rotational or pseudo-rotational slides.
	Topples	0	
	Slides	6	
	Spreads	0	
	Flows	0	
Material	Earth	8	Mainly applicable to landsliding involving earth and debris. Applicability in rock limited by difficulty of excavation.
	Debris	6	
	Rock	2	
Depth of movement	Superficial (< 0.5 m)	6	Typically applicable to relatively small and/or shallow landslides. It is generally impractical and not necessary to carry out large scale substitution as would be necessary on large landslides.
	Shallow (0.5 to 3 m)	6	
	Medium (3 to 8 m)	6	
	Deep (8 to 15 m)	4	
	Very deep (> 15 m)	0	
Rate of movement (Varnes, 1978)	Moderately to fast	0	While excavation can be carried out without special difficulty when the rate of movement is slow (5 cm/day) or less, backfilling with lightweight fill presupposes that the slide is stable or moving at most very slowly.
	Slow	2	
	Very slow	6	
	Extremely slow	8	
Groundwater	Artesian	4	High or artesian groundwater conditions pose special problems both to the excavation and to the stability of the slope after backfilling with lightweight fill, limiting the applicability of this technique when these conditions occur.
	High	6	
	Low	8	
	Absent	8	
Surface water	Rain	6	Surface flows must be diverted to prevent them from accumulating in the lightweight fill and/or infiltrating the portion of the landslide mass left in place. Drainage to be provided both on surface and at interface between fill and natural soil.
	Snowmelt	6	
	Localized	6	
	Stream	2	
	Torrent	0	
	River	0	
Maturity		6	Concept is well developed but knowledge of mechanical properties and applicability of different lightweight fills still not fully established.
Reliability		6	The reliability of the technique depends on the evaluation of the stability of the treated slope and on the homogeneity and durability of the fill used.
Implementation		6	Can be implemented with widely available equipment. Possible difficulties with excavation in rock and with the disposal of arisings. Construction control.
Typical Cost		6	Moderate to high, depending on the material used.

Note
Ratings are given on a scale of 1 to 10; the higher the grade, the most suitable is the specific method under consideration to use in landslides of the given characteristics, evaluated individually. Overall suitability to specific case under consideration may be obtained by a weighted average of these ratings, with user defined weights. Zero rating means "not applicable"

MITIGATION THROUGH REDUCTION OF HAZARD

2 MODIFYING THE SLOPE GEOMETRY/MASS DISTRIBUTION

2.4 SUBSTITUTION OF MATERIAL IN DRIVING AREA WITH LIGHTWEIGTH FILL

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MITIGATION THROUGH REDUCTION OF HAZARD

2 MODIFYING THE SLOPE GEOMETRY/MASS DISTRIBUTION

2.5 ADDITION OF MATERIAL TO THE AREA MAINTAINING STABILITY

Description

The addition of material to the toe or resisting area (or more in general, buttressing, counterweight fills and toe berms) operates by increasing the resisting forces, thereby improving overall slope stability, by providing sufficient dead weight or restraint near the toe of the unstable slope (Figure 1).

This method is most suitable in cases where the instability mechanism occurs as a rotational or pseudo-rotational slide, e.g. where the displaced mass moves as a relatively coherent mass along a spoon-shaped (curved upward) failure surface with little internal deformation. It is generally ineffective on translational slides on long, uniform planar slopes, or on flow-type landslides.

Generally it is most practical on small slumps or small rotational failures, but several examples exist where this technique has been applied successfully on large landslides where conditions allowed large scale earthmoving to be carried out. In these cases, this mitigation measure is typically supplemented by drainage and/or other mitigation measures.

It should always be kept in mind that when fill is placed on the landslide body itself, the driving forces are also increased. It is therefore necessary to locate the fill in such manner that the increase in resisting forces exceeds the increase in driving forces. This is typically achieved by placing the fill at or very near the toe of the landslide. The neutral line concept, described in fact sheet 2.0 on “mitigation by modifying the slope geometry / mass distribution; general aspects” can be used for a preliminary evaluation of the relative merits of the proposed fill.

lt is worth noting that the increase in resisting forces associated with the fill will tend to increase in time as a result of the increase in normal effective stress on the failure surface as consolidation takes place. Thus, the most critical conditions typically occur during construction, when the Contractor is still on site and it is easier to respond to unexpected performance.

Butress fills are normally constructed of blasted quarry rock, boulders and cobbles and coarse gravel fill, which are relatively free draining. If fine grained material is used, it is essential to include a drainage layer at the interface between the buttress and the underlying natural soil (Figure 2)

This technique can be incorporated economically in highway or railway projects if it is possible to design the alignment to match the stabilization requirements, as was done for example with the Taren Landslide (Kelly and Martin, 1985)

The main limitations of the technique relate to the following issues:

- Filling may actually destabilize the ground farther down-slope;
- Satisfactory solutions may involve significant modification of the landscape and possible interference with water courses at the toe of the landslide;
- Filling may require large volumes of material, to be procured off-site; availability of suitable fill may limit application of this technique;
- Filling may interfere with existing structures and services; this is potentially significant when considering this type of mitigation for “potential” landslides, while on actual landslides the residual value of existing structures and facilities can be very low;
- Filling on or at the toe of active landslides requires special care to ensure the safety of workers; in particular, it is necessary to assess the possibility of sudden accelerations and to have in place well drilled evacuation plans.

Examples of large landslides stabilized by this technique (alone or in combination with other mitigation measures are provided by Gedney and Weber (1978); Edil (1992); Kropp and Thomas (1992). Figures 3 shows a similar example described by Millet et al. (1992).

Design

For general considerations on the geotechnical design of mitigation by addition of material to the resisting area, reference shall be made to the general fact sheet 2.0 on hazard mitigation by changes in slope geometry and/or mass distribution.

The basic design of buttress fills is similar to the design for external stability of conventional gravity retaining structures, including check of the following limiting situations, evaluated taking into account the loading induced by the landslide body.

- Overturning
- Sliding at or below the base
- Bearing capacity of the foundations, including evaluation of the stability of the slope downhill of the buttress

It is also necessary to evaluate the possibility that the landslide body overrides the buttress, especially on slides with a significant translational component.

Possible internal failure modes should also be checked to ensure that the buttress does not fail by shear.

Figure 1: Coarse grained or rock buttress for slope stabilization (source: Gedney and Weber, 1978)

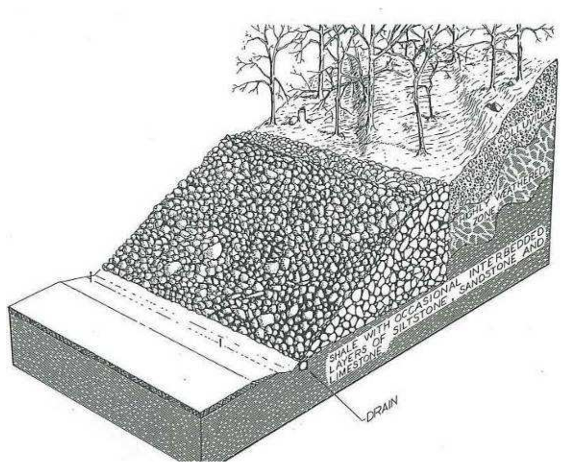


Figure 2: Drainage layer at interface between natural soil and low permeability fill (source: Millet et al., 1992)

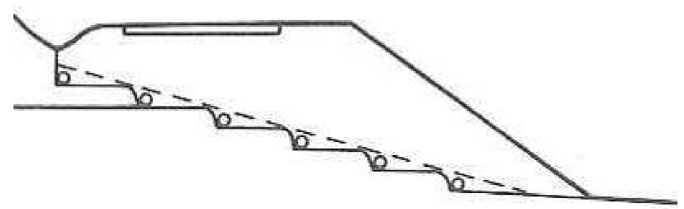
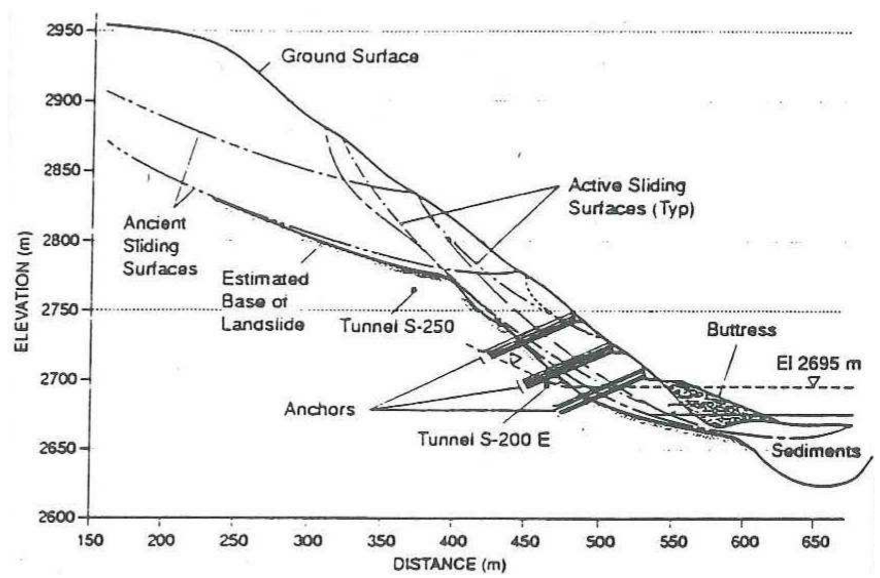


Figure 3: Stabilization of Tablachaca Dam Landslide, Peru, crossection (source: Millet et al., 1992)



MITIGATION THROUGH REDUCTION OF HAZARD			
2 MODIFYING THE SLOPE GEOMETRY/MASS DISTRIBUTION			
2.5 ADDITION OF MATERIAL TO THE AREA MAINTAINING STABILITY			
APPLICABILITY			
Class	**Descriptor**	**Rating**	**Notes**
Type of movement (Cruden & Varnes, 1996)	Falls	0	Most suited to rotational or pseudo-rotational slides; may be useful to reduce toppling hazard in certain conditions.
	Topples	2	
	Slides	8	
	Spreads	0	
	Flows	0	
Material	Earth	8	Mainly applicable to landsliding involving earth and debris. Applicability in rock limited by typical slope geometry and failure mode.
	Debris	6	
	Rock	4	
Depth of movement	Superficial (< 0.5 m)	6	Typically applicable to relatively small and/or shallow landslides. The implications of large scale filling and procurement typically make this technique impractical for deep and very deep slides. On the other hand, it may be the only suitable technique in very large landslides, besides drainage.
	Shallow (0.5 to 3 m)	8	
	Medium (3 to 8 m)	8	
	Deep (8 to 15 m)	6	
	Very deep (> 15 m)	4	
Rate of movement (Varnes, 1978)	Moderately to fast	2	Can be carried out without special difficulty when the rate of movement is slow (5 cm/day) or less.
	Slow	8	
	Very slow	8	
	Extremely slow	8	
Groundwater	Artesian	8	Applicable in all groundwater conditions. Adequate drainage must be provided at the interface between low permeability fills and natural soil.
	High	8	
	Low	8	
	Absent	8	
Surface water	Rain	6	Possible limitations in applying this technique where the landslide is caused by or impinges on a water course, although examples exist where rivers have been diverted to implement this type of solution. Adequate protection must be provided in this case against toe erosion by wave or current.
	Snowmelt	6	
	Localized	4	
	Stream	2	
	Torrent	0	
	River	0	
Maturity		10	Simple technique. Potential benefits and limits of applicability are well established.
Reliability		10	The reliability of the technique depends on the reliability of the evaluation of the stability of the treated slope. More reliable than excavation.
Implementation		8	Easily implemented with widely available equipment. Possible difficulties with the procurement and/or control of compaction of fill.
Typical Cost		8	Moderate, provided the work does not involve diversion of major water courses or interference with existing infrastructure.

Note
Ratings are given on a scale of 1 to 10; the higher the grade, the most suitable is the specific method under consideration to use in landslides of the given characteristics, evaluated individually. Overall suitability to specific case under consideration may be obtained by a weighted average of these ratings, with user defined weights. Zero rating means "not applicable"

MITIGATION THROUGH REDUCTION OF HAZARD

2 MODIFYING THE SLOPE GEOMETRY/MASS DISTRIBUTION

2.5 ADDITION OF MATERIAL TO THE AREA MAINTAINING STABILITY

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MITIGATION THROUGH REDUCTION OF HAZARD

FACT SHEET 3

MODIFYING THE SURFACE WATER REGIME – SURFACE DRAINAGE

MITIGATION THROUGH REDUCTION OF HAZARD

3 MODIFYING THE SURFACE WATER REGIME – SURFACE DRAINAGE

3.0 GENERAL

Description

Surface water features such as streams, lakes, springs, seeps, marshes and closed topographic depressions are of importance to slope stability. Springs and seeps near the crest of a slope can supply recharge zones that provide ground water to the actually or potentially unstable slopes; springs and seeps near the base of a slope indicate discharge zones that can help in estimating the piezometric surfaces in the slope; localized closed topographic depressions on slopes (Figure 1) are usually zones of ground water recharge, particularly if ground cracks are present in or adjacent to them; surface water infiltrating into actually or potentially unstable zones through cracks and fissures may activate or reactivate the landslide.

Good surface drainage is strongly recommended as part of the treatment of any actual or potential landslide (Cedergren, 1989). Modifying the surface (this section) and subsurface (see section 4) water regime increases slope stability helping to prevent potential landslides or to mitigate existing ones.

Surface drainage measures operate to achieve the following objectives:

- They reduce the surface water ponding on or flowing across the face of the slope;
- They reduce the amount of surface water thar can infiltrate into the ground;
- They modify the hydraulic regime of natural streams or river channels.

Achieving these objectives helps prevent erosion of the face and minimize the tendency for localized failures on the slope.

Ditches, channels, pipework, etc., are widely used to achieve the first objective, especially in situations where large volumes of runoff are anticipated (Figure 2).

Local regrading, impermeabilization, sealing tension cracks, geomembranes, impervious facing, vegetation (hydrological effect) are largely used to achieve the second objective.

Hydraulic control works and diversion channels are used to achieve the third objective.

Special care is necessary when dealing with landslides in built-up environments, since roads, drains and other buried surfaces may cause significant adverse changes to the drainage regime of the area, modifying the effective geometry and extent of the catchment area and/or ampltyfying and accelerating run off. In these situation, leakege from existing drains, acqueducts cess-pits etc should be addressed as part of an integrated approach to slope drainage.

Design

Surface drainage measures normally require minimal engineering design; it involves expertise in hydrology, to determine anticipated flows, and hydraulic engineering, to verify the adequacy of the design.

Surface drainage works require frequent maintenance. The Geotechnical Controll Office of Hong Kong (Geotechnical Control Office, 1984) has presented useful guidelines for maintenance of surface drainage systems; the guidelines particularly recommend the use of surface channels (ditches) as opposed to pipes placed on the surface.

References:

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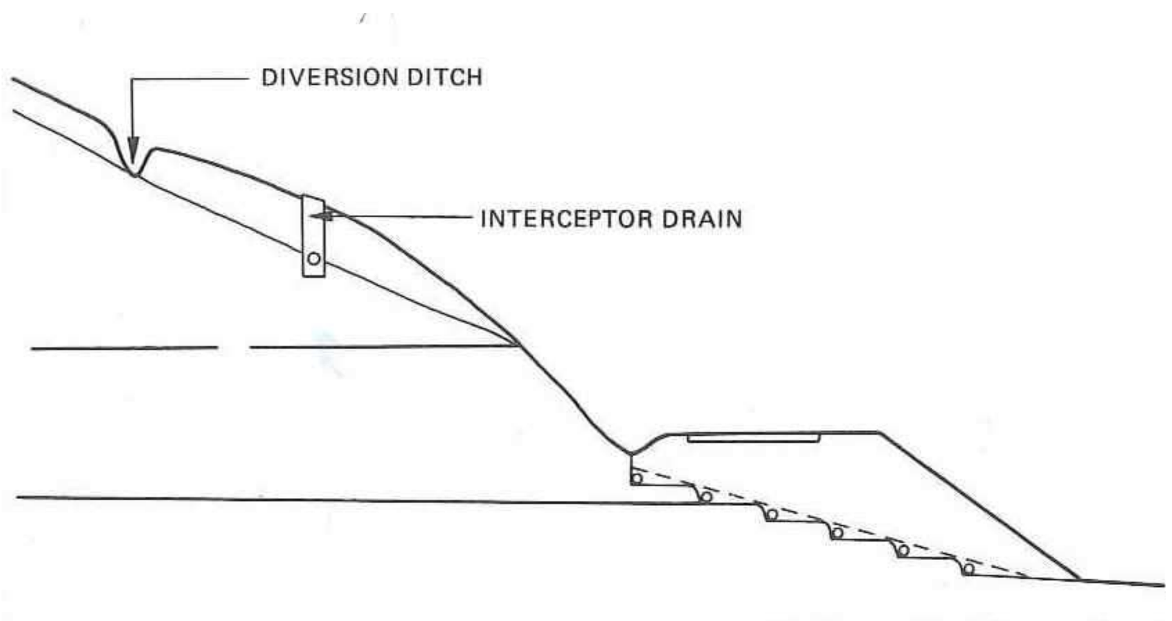
Figure 1: Ponding in topography disturbed by landsliding, Lillaz, nr. Cogne (AO), Italy (photo: G. Vaciago, SGI-MI)

Figure 2: Coarse grained or rock buttress for slope stabilization (source: Gedney and Weber, 1978)



MITIGATION THROUGH REDUCTION OF HAZARD

3 MODIFYING THE SURFACE WATER REGIME – SURFACE DRAINAGE

3.1 SURFACE DRAINAGE WORKS (DITCHES, CHANNELS, PIPEWORK)

Description

Surface drainage works are used to collect and direct surface runoff in a controlled manner, to minimize the quantity of surface water flowing into actually or potentially unstable slopes.
Surface drainage works are especially important at the head of the slope to intercept the run-off and reduce the surface water flowing downstream across the face of the slope. This may be achieved by open ditches at the head of the slope.
Ditches on the main landslide body are used to dispose of local surface runoff and any water arising from deep drainage works.
Different types of ditches are used to drain surface runoff. The cross-section of ditches is usually trapezoidal, although small ones may be V or U-shaped or semicircular; their dimensions vary according to the expected runoff, the need for open water storage, the risk of bank erosion, the need to accommodate the transit of construction or maintenance equipment and the available means for maintenance (Figures 1 and 2 and Table 1).

Table 1: Typical dimensions of open ditches (ftp://ftp.fao.org/dicrep/fao/010/a0975e/a0975e.pdf)

Type of ditch	Depth (m)	Bed width (m)	Side slope (v:h)	Maximum side slope (v: h)
V-shaped	0.3 to 0.6	-	1: 6	1:3
V-shaped	>0.6	-	1:4	1:3
Trapezoidal	0.3 to 1	As required	1:4	1:2

Ditch gradient should be at least 2% to ensure rapid flow away from the potentially unstable areas and to promote self cleaning from any windblown or other debris that would tend to accumulate, causing local blockage and spillage.
Ditches should be lined to minimize erosion and uncontrolled infiltration. The lining may consist of cast-in-place or prefabricated concrete, pitched stone (Figure 3), rip rap, gabion mattresses or baskets, speciality geotextiles or geocomposites, zinc coated steel or PVC half-pipes. Flexible, self-healing lining or pipes should be used in areas susceptible to cracking and movements.
Where permeable linings are used, this should be in association with an impermeable geomembrane to minimize infiltration. Geomembranes may also be used by themselves for temporary or emergency applications, but they are easily damaged by wind and direct sunlight and should not normally be used by themselves for permnnent applications.
Techniques must be adapted to ground conditions and local technology; an example is provided by Anderson and Holcombe (2004; 2008) who describe the development and application at community level of good drainage practices with locally available, affordable technologies in St Lucia, West Indies, consisting of ditches lined with a specialised plastic, held in place by a wire mesh (Figure 4).

Design

Ditches must have enough capacity to transport the drainage water in wet period; however they are sometimes made wider than needed in order to create more storage in the open water system. Such temporary storage is a good way of diminishing the peak outflow from the area, as occurs after heavy rains. Thus it reduces the required capacity of downstream constructions, such as the larger watercourses, culverts and pumpung stations.
Ditches are often relatively unaccessible and may receive less maintenance than would be appropriate. Accordingly, it is advisable to design them with a generous freebord to minimize the risk of blockage and spilling.
Steps or other energy dissipation systems should be used on and at the toe of steep sections, to prevent excessive flow speeds and the resulting erosion.

Figure 1: Typical arrangement of open ditches
(source: ftp.fao.org.pdf)

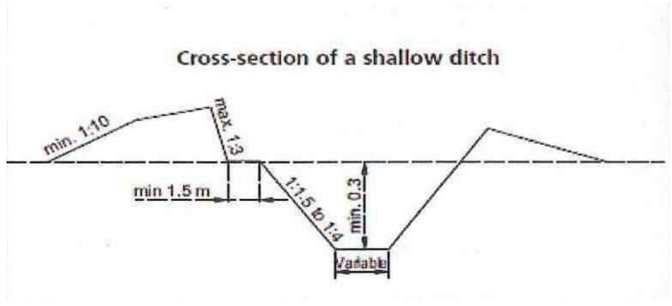


Figure 2: Typical detailing of open ditches
(source: www.co.nrcs.usda.gov)

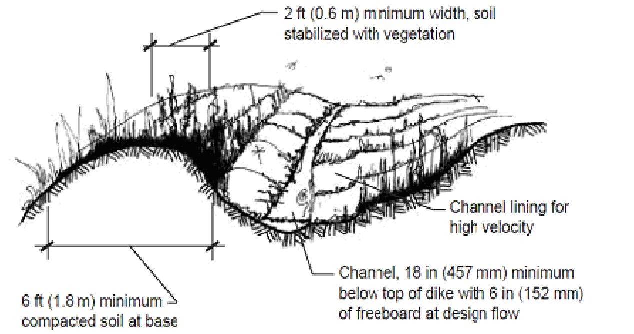


Figure 3: Open ditch lined with pitched stone, Gimillan nr. Cogne (AO), Italy (photo: G. Vaciago, SGI-MI)

Figure 4: STAR™ drainage system installed by residents in St Lucia, West Indies
(source: Anderson and Holcombe, 2008)

MITIGATION THROUGH REDUCTION OF HAZARD

3 MODIFYING THE SURFACE WATER REGIME – SURFACE DRAINAGE

3. 1 SURFACE DRAINAGE WORKS (DITCHES, CHANNELS, PIPEWORK)

APPLICABILITY

Class	Descriptor	Rating	Notes
Type of movement (Cruden & Varnes, 1996)	Falls	0	Most suited to all types of slides and, subject to circumstances in flows. In spreads, only useful as remediation, not as a preventive measure.
	Topples	0	
	Slides	8	
	Spreads	4	
	Flows	6	
Material	Earth	8	Mainly applicable to landsliding involving earth and debris. Applicability in rock limited by typical slope geometry and failure mode. Potential difficulties in excavation and impermeabilization of ditches in coarse debris.
	Debris	6	
	Rock	2	
Depth of movement	Superficial (< 0.5 m)	8	Typically applicable to landslides of any depth, but relative effectiveness decreases with increasing depth of movement.
	Shallow (0.5 to 3 m)	8	
	Medium (3 to 8 m)	6	
	Deep (8 to 15 m)	4	
	Very deep (> 15 m)	0	
Rate of movement (Varnes, 1978)	Moderately to fast	0	Can be carried out without special difficulty when the rate of movement is slow (5 cm/day) or less, but may be disrupted and will require additional maintenance or reconstruction as a result of continued movement.
	Slow	6	
	Very slow	8	
	Extremely slow	8	
Groundwater	Artesian	6	Applicable irrespective of groundwater conditions. It does not drain groundwater. Effects on groundwater levels only indirect through reduced infiltration.
	High	6	
	Low	6	
	Absent	6	
Surface water	Rain	8	See fact sheet 3.7 for diversion channels for main water courses.
	Snowmelt	8	
	Localized	8	
	Stream	4	
	Torrent	0	
	River	0	
Maturity		10	Simple technique. Potential benefits and limits of applicability are well established.
Reliability		8	Effects on stability only indirect. The reliability in the long term may be impaired by further movement or poor maintenance.
Implementation		10	Easily implemented with widely available equipment.
Typical Cost		10	Low, where applicable.

Note
Ratings are given on a scale of 1 to 10; the higher the grade, the most suitable is the specific method under consideration to use in landslides of the given characteristics, evaluated individually. Overall suitability to specific case under consideration may be obtained by a weighted average of these ratings, with user defined weights. Zero rating means "not applicable"

MITIGATION THROUGH REDUCTION OF HAZARD	
3 MODIFYING THE SURFACE WATER REGIME – SURFACE DRAINAGE	
3. 1 SURFACE DRAINAGE WORKS (DITCHES, CHANNELS, PIPEWORK)	
References: Anderson M.G., Holcombe E.A. (2004). “Management of slope stability in communities” Insight, Vol 6, 15-17. Anderson M.G., Holcombe E.A. (2008). “A new sustainable landslide risk reduction methodology for communities in lower income countries”. In: Proc. of the First World landslide Forum, Tokyo, Parallel Session Volume, 61-64.	

MITIGATION THROUGH REDUCTION OF HAZARD

3 MODIFYING THE SURFACE WATER REGIME – SURFACE DRAINAGE

3.2 LOCAL REGRADING TO FACILITATE RUN-OFF

Description

Smoothing the topography of the slope surface can prevent surface water from ponding in local depressions (Figure 1), thus reducing the opportunity for infiltration. Any depressions on the slope that might retain standing water should be removed by regrading, infilling and exacavation works (Figure 2), combined with surface and/or shallow drainage (fact sheets 3.1 and 4.1), measures to promote rapid runoff (fact sheet 3.4) and measures to reduce net rainfall thanks to canopy storage in vegetation (fact sheet 3.5).

Regrading inevitably damages the residual vegetation cover, which should be reinstated without delay to minimize erosion. Reference may be made to fact sheet 1 for details.

Local regrading as described here should not be confused with general modification of the slope geometry, described in Section 2.

Design

The design should balance cut and fill, in order to minimize the cost and environmental impact of the works.

The design should minimize major changes to the mass distribution of the slope, unless this is carried out deliberately as part of the stabilization works in accordance with the principles and methods described in fact sheets 2.

Figure 1: Ponding in topography disturbed by landsliding, Lillaz, nr. Cogne (AO), Italy (photo: G. Vaciago SGI-MI)

Figure 2: Stabilization of Lillaz landslide, nr. Cogne (AO), Italy (photo: G. Vaciago SGI-MI)

MITIGATION THROUGH REDUCTION OF HAZARD			
3 MODIFYING THE SURFACE WATER REGIME – SURFACE DRAINAGE			
3.2 LOCAL REGRADING TO FACILITATE RUN-OFF			
APPLICABILITY			
Class	**Descriptor**	**Rating**	**Notes**
Type of movement (Cruden & Varnes, 1996)	Falls	0	Most suited to all types of slides and, subject to circumstances in flows. In spreads, only useful as remediation, not as a preventive measure.
	Topples	0	
	Slides	8	
	Spreads	4	
	Flows	6	
Material	Earth	8	Mainly applicable to landsliding involving earth and debris. Applicability in rock limited by typical slope geometry and failure mode. Potential difficulties in excavation and impermeabilization of ditches in coarse debris.
	Debris	6	
	Rock	2	
Depth of movement	Superficial (< 0.5 m)	8	Typically applicable to landslides of any depth, but relative effectiveness decreases with increasing depth of movement.
	Shallow (0.5 to 3 m)	8	
	Medium (3 to 8 m)	6	
	Deep (8 to 15 m)	4	
	Very deep (> 15 m)	0	
Rate of movement (Varnes, 1978)	Moderately to fast	2	Can be carried out without special difficulty when the rate of movement is slow (5 cm/day) or less, but may be disrupted and will require additional maintenance or reconstruction as a result of continued movement. May be applicable, with special precautions and limited effectiveness due to continuous disruption, to moderately fast movements.
	Slow	6	
	Very slow	8	
	Extremely slow	8	
Groundwater	Artesian	6	Applicable irrespective of groundwater conditions. It does not drain groundwater. Effects on groundwater levels only indirect through reduced infiltration. Potential difficulties in carrying out regarding in areas of high or artesian groundwater levels, depending on the depth of local excavation required.
	High	6	
	Low	8	
	Absent	8	
Surface water	Rain	8	See fact sheet 3.7 for diversion channels for main water courses.
	Snowmelt	8	
	Localized	8	
	Stream	4	
	Torrent	0	
	River	0	
Maturity		10	Simple technique. Potential benefits and limits of applicability are well established.
Reliability		8	Effects on stability only indirect. The reliability in the long term may be impaired by further movement or poor maintenance.
Implementation		10	Easily implemented with widely available equipment.
Typical Cost		10	Low, where applicable.

Note
Ratings are given on a scale of 1 to 10; the higher the grade, the most suitable is the specific method under consideration to use in landslides of the given characteristics, evaluated individually. Overall suitability to specific case under consideration may be obtained by a weighted average of these ratings, with user defined weights. Zero rating means "not applicable"

MITIGATION THROUGH REDUCTION OF HAZARD	
3 MODIFYING THE SURFACE WATER REGIME – SURFACE DRAINAGE	
3.2 LOCAL REGRADING TO FACILITATE RUN-OFF	
References:	

MITIGATION THROUGH REDUCTION OF HAZARD

3 MODIFYING THE SURFACE WATER REGIME – SURFACE DRAINAGE

3.3 SEALING TENSION CRACKS

Description

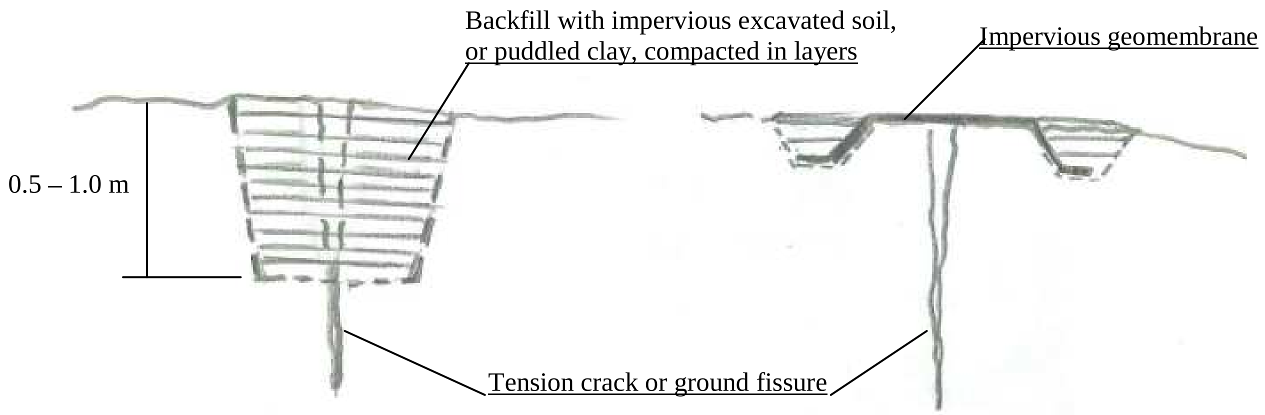
In the context of landslide mitigation, iIt is always necessary and beneficial to prevent the infiltration of surface water into the ground in or close to existing or potential landslides. Good surface drainage is therefore necessary in these areas. This is particularly significant where failure has already occurred, because the runoff water may flow into cracks and fissure in or at the boundary of the unstable soil mass, especially at tension cracks at the head of the slope (Figure 1). This would give rise to the following unfavourable effects:

- Raise piezometric levels in the unstable mass, reducing effective stress and consequently shera strength on the slip surface;
- Provide additional driving force by means of the hydrostatic pressure of free water in tension cracks at the head of the slide.

The most common methods for sealing cracks (Figure 2) consists of filling them with puddle clay or other impervious fill; it is often sufficient to excavate a trench along the tension crack and to backfill it with the excavated impervious material, possibly adding small quantities of bentonite or other natural material to reduce permeability further, and shaping the ground so that surface water does not pond in the area. If necessary, an impervious membrane may be added at or near the surface
Impervious membranes may be used by themselves as an emergency or temporary measure, while arrangements are being made for the works to be carried out.
Regular inspection and maintenance is required in case of continued movement, since it may cause previously sealed cracks to reopen.

Figure 1: Tension cracks, 2009 reactivation, Petacciato Landslide, Italy (source: SGI-MI Project files)

Figure 2: Sealing tension cracks – typical detail



MITIGATION THROUGH REDUCTION OF HAZARD			
3 MODIFYING THE SURFACE WATER REGIME – SURFACE DRAINAGE			
3.3 SEALING TENSION CRACKS			
APPLICABILITY			
Class	**Descriptor**	**Rating**	**Notes**
Type of movement (Cruden & Varnes, 1996)	Falls	0	Most suited to all types of slides. In spreads, only useful as remediation, not as a preventive measure.
	Topples	0	
	Slides	8	
	Spreads	4	
	Flows	0	
Material	Earth	8	Mainly applicable to landsliding involving earth and only to a lesser extent in debris. Applicability in rock limited by typical slope geometry and failure mode, but note that in deep seated rock slides tension cracks propagating through the surface cover would also benefit.
	Debris	6	
	Rock	2	
Depth of movement	Superficial (< 0.5 m)	8	Typically applicable to landslides of any depth, but relative effectiveness decreases with increasing depth of movement.
	Shallow (0.5 to 3 m)	8	
	Medium (3 to 8 m)	6	
	Deep (8 to 15 m)	4	
	Very deep (> 15 m)	0	
Rate of movement (Varnes, 1978)	Moderately to fast	2	Can be carried out without special difficulty when the rate of movement is slow (5 cm/day) or less, but may be disrupted and will require additional maintenance or reconstruction as a result of continued movement. May be applicable, with special precautions and limited effectiveness due to continuous disruption, to moderately fast movements.
	Slow	6	
	Very slow	8	
	Extremely slow	8	
Groundwater	Artesian	6	Applicable irrespective of groundwater conditions. Effects on groundwater levels only indirect through reduced infiltration. Potential difficulties in carrying out in areas of high or artesian groundwater levels, depending on the depth of local excavation required.
	High	6	
	Low	8	
	Absent	8	
Surface water	Rain	8	Water courses should be diverted.
	Snowmelt	8	
	Localized	8	
	Stream	4	
	Torrent	0	
	River	0	
Maturity		10	Simple technique. Potential benefits and limits of applicability are well established.
Reliability		8	Effects on stability only indirect. The reliability in the long term may be impaired by further movement or poor maintenance.
Implementation		10	Easily implemented with widely available equipment.
Typical Cost		10	Low, where applicable.

Note
Ratings are given on a scale of 1 to 10; the higher the grade, the most suitable is the specific method under consideration to use in landslides of the given characteristics, evaluated individually. Overall suitability to specific case under consideration may be obtained by a weighted average of these ratings, with user defined weights. Zero rating means "not applicable"

MITIGATION THROUGH REDUCTION OF HAZARD	
3 MODIFYING THE SURFACE WATER REGIME – SURFACE DRAINAGE	
3.3 SEALING TENSION CRACKS	
References:	

MITIGATION THROUGH REDUCTION OF HAZARD

3 MODIFYING THE SURFACE WATER REGIME – SURFACE DRAINAGE

3.4 IMPERMEABILIZATION (GEOMEMBRANES, IMPERVIOUS FACING)

Description

In the context of landslide mitigation, it is always necessary and beneficial to prevent the infiltration of surface water into the ground in or close to existing or potential landslides. Good surface drainage is therefore necessary in these areas to avoid rises in piezometric levels in the unstable mass, reducing effective stress and consequently shear strength on the slip surface.

Impervious membranes are normally used as a short term, temporary or emergency measure.

Impervious facing is normally used as a permanent measure as part of landslide remediation or as a preventive measure on excavated slopes (see section 1).

Vegetation may be considered to provide partial impermeabilization through the canopy storage effects (see fact sheet 3.5).

MITIGATION THROUGH REDUCTION OF HAZARD			
3 MODIFYING THE SURFACE WATER REGIME – SURFACE DRAINAGE			
3.4 IMPERMEABILIZATION (GEOMEMBRANES, IMPERVIOUS FACING)			
APPLICABILITY			
Class	**Descriptor**	**Rating**	**Notes**
Type of movement (Cruden & Varnes, 1996)	Falls	0	Most suited to all types of slides. In spreads, only useful as remediation, not as a preventive measure.
	Topples	0	
	Slides	8	
	Spreads	4	
	Flows	0	
Material	Earth	8	Mainly applicable to landsliding involving earth and only to a lesser extent in debris. Applicability in rock limited by typical slope geometry and failure mode, but note that in deep seated rock slides tension cracks propagating through the surface cover would also benefit.
	Debris	6	
	Rock	2	
Depth of movement	Superficial (< 0.5 m)	8	Typically applicable to landslides of any depth, but relative effectiveness decreases with increasing depth of movement.
	Shallow (0.5 to 3 m)	8	
	Medium (3 to 8 m)	6	
	Deep (8 to 15 m)	4	
	Very deep (> 15 m)	0	
Rate of movement (Varnes, 1978)	Moderately to fast	2	Can be carried out without special difficulty when the rate of movement is slow (5 cm/day) or less, but may be disrupted and will rquire additional maintenance or reconstruction as a result of continued movement. May be applicable, with special precautions and limited effectiveness due to continuous disruption, to moderately fast movements.
	Slow	6	
	Very slow	8	
	Extremely slow	8	
Groundwater	Artesian	6	Applicable irrespective of groundwater conditions. Effects on groundwater levels only indirect through reduced infiltration. Potential difficulties in carrying out in areas of high or artesian groundwater levels, depending on the depth of local excavation required.
	High	6	
	Low	8	
	Absent	8	
Surface water	Rain	8	Water courses should be diverted.
	Snowmelt	8	
	Localized	8	
	Stream	4	
	Torrent	0	
	River	0	
Maturity		10	Simple technique. Potential benefits and limits of applicability are well established.
Reliability		8	Effects on stability only indirect. The reliability in the long term may be impaired by further movement or poor maintenance.
Implementation		10	Easily implemented with widely available equipment.
Typical Cost		10	Low, where applicable.

Note
Ratings are given on a scale of 1 to 10; the higher the grade, the most suitable is the specific method under consideration to use in landslides of the given characteristics, evaluated individually. Overall suitability to specific case under consideration may be obtained by a weighted average of these ratings, with user defined weights. Zero rating means "not applicable"

MITIGATION THROUGH REDUCTION OF HAZARD

3 MODIFYING THE SURFACE WATER REGIME – SURFACE DRAINAGE

3.4 IMPERMEABILIZATION (GEOMEMBRANES, IMPERVIOUS FACING)

References:

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MITIGATION THROUGH REDUCTION OF HAZARD

3 MODIFYING THE SURFACE WATER REGIME – SURFACE DRAINAGE

3. 5 VEGETATION – HYDROLOGICAL EFFECTS

Description

Both soil erosion and shallow landslides may occur due to degradation or removal of land cover or vegetation. The use of vegetation to mitigate these phenomena has long been common practice, The role of vegetation in mitigating soil erosion is described in section 1 and summarized in Figure 1. The role of vegetation in mitigating shallow landslides is twofold: vegetation contributes to slope stability through root reinforcement (mechanical function, discussed in fact sheet 5.1) and through rainfall interception and evapotranspiration (hydrological function, discussed here).

Greenwood et al. (2007) highlight that vegetation may also result in increased suction (negative pore pressure) in unsaturated soil, potentially increasing the apparent cohesion of the soil.

Vegetation is widely believed to improve the stability of slopes, especially on steep slopes and with respect to superficial or shallow movements. However, it can take a long time to become effective at depth and it can also have negative effects, as summarized in Table 1 (Greenway 1987; Wu, 1995).

Table 1: Mechanical and hydrogeological effects of vegetation on slope stability (Wu, 1995)

Process	Type	Effect on stability
1. Roots increase permeability, increase infiltration, and thereby increase pore pressure	Hydrological	Negative
2. Vegetation increases interception and evapotranspiration, and thereby reduce pore pressure	Hydrological	Positive
3. Vegetation increases weight or surcharge, and thereby increases load on slope	Mechanical	Negative
4. Vegetation increases wind resistance, and thereby increases load on slope	Mechanical	Negative
5. Roots reinforce soil, and thereby increase strength	Mechanical	Positive

Information on the mechanical and hydrological effects of vegetation is provided by the Hong Kong Geotechnical Manual for Slopes (Geotechnical Control Office, 1984), reflecting one of the most comprehensive research programs in the world on the engineering role of vegetation for slope stabilization (Barker, 1991)

The hydrological function of vegetation influences the rate of water flow into and on the slope through the process of interception stem flow, leaf drip evaporation, evapotranspiration, infiltration, etc., which may reduce pore water pressures in the ground.

Canopy interception is the loss of available precipitation due to storage and evaporation from the canopy. For closed canopy forests, the interception ranges from 10 to 50% of the total precipitation. The capacity of the canopy to intercept and store water differs among the ecosystems. It depends mainly on the leaf surface area. Conifer (needle leaved, e.g. pine) forests store around 15% of the precipitation, whereas deciduous (broad leaved) forest store from 5 to 10% of the precipitation.

The bark structure and architecture of stems and trunks influence the amount and direction of stemflow (water movement from stems to the ground). Trees and shrubs with small barks have greater stemflow (around 12% of the precipitation) than the rough-barked plants, such as conifer which release around 2% of the precipitation received by their stem (Chapin et al., 2002).

Collison and Anderson (1996) developed a vegetation cover model (Figure 2). In the model, canopy interception is calculated by a subroutine which include these parameters.

- Canopy area per cell (m^2/m^2)
- Leaf index ratio (m^2/m^2)
- Maximum depth of canopy store (m)
- Stemflow rate (percent rainfall)
- Maximum evaporation rate (m/s).

In the model rainfall is stored in the canopy up to a maximum depth, beyond which throughfall occurs. Rain stored in the canopy is subject to evaporation, which is a function of time of a day.

Infiltration of rainwater into the top soil depends on hydraulic conductivity and preferential flow through macropores created by cracks in the soils and channels produced by plant roots and soil organisms. Collison and Anderson (1996) tested significant increase in permeability due to roots and found that permeability of top soil depends on root density.

The hydrological effect of the canopy (canopy interception, stemflow and evaporation) on slope stability is controversial and presumably depends on the specifics of each case. For example:

- Collison and Anderson (1996) concluded that in high intensity rain storm areas, canopy interception, stemflow and evaporation have little effect on slope stability.
- A case study conducted in riparian vegetation cover for riverbank stability suggests that hydrological effects of trees contributed 29%, while grasses contributed 15% to the total Factor of Safety (Simon and Collison, 2002).

Figure 1: Some influences of vegetation on soil stability
(source: Coppin and Richards, 1990)

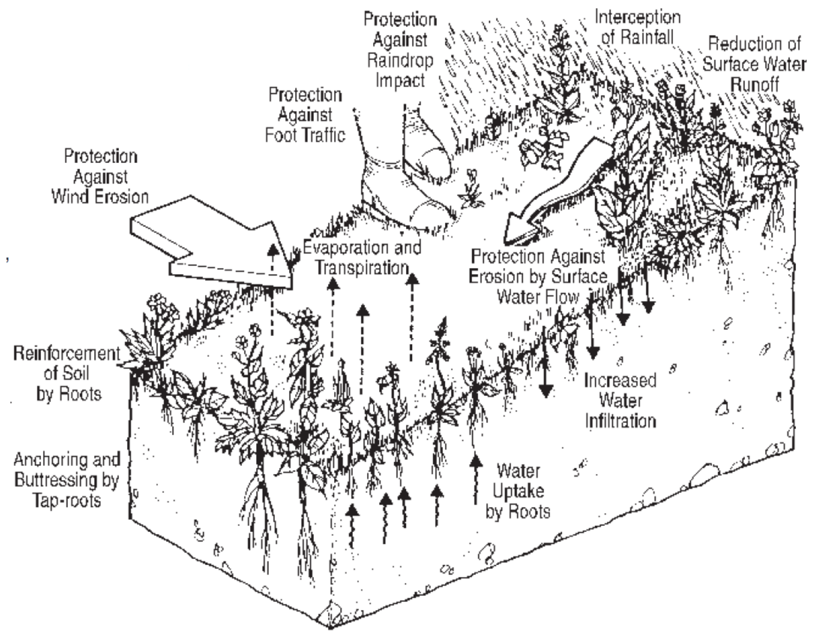
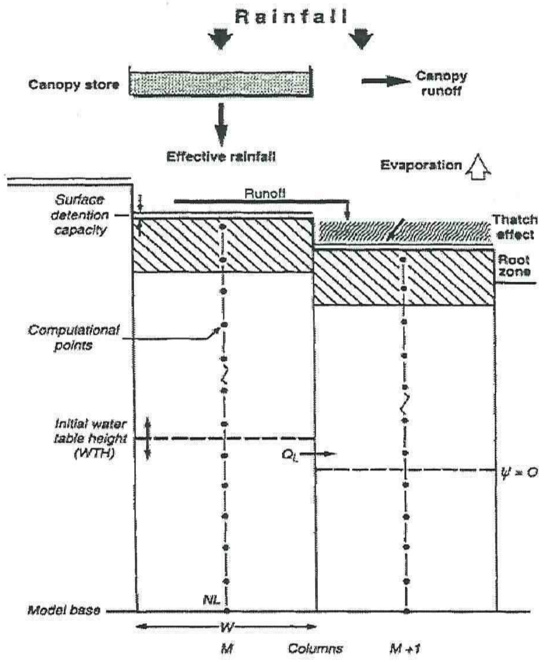


Figure 2: Model of hydrological and mechanical functions of vegetation
(source: Collison and Anderson, 1996)



MITIGATION THROUGH REDUCTION OF HAZARD			
3 MODIFYING THE SURFACE WATER REGIME – SURFACE DRAINAGE			
3. 5 VEGETATION – HYDROLOGICAL EFFECTS			
APPLICABILITY			
Class	**Descriptor**	**Rating**	**Notes**
Type of movement (Cruden & Varnes, 1996)	Falls	0	Most suited to all types of slides and, to a lesser extent, flows, by attenuating the impact of intense precipitation and by inducing suctions.
	Topples	0	
	Slides	8	
	Spreads	0	
	Flows	6	
Material	Earth	8	Applicable to landsliding involving earth and only to a lesser extent in debris. Applicability in rock limited by typical slope geometry and failure mode.
	Debris	6	
	Rock	0	
Depth of movement	Superficial (< 0.5 m)	8	Typically applicable to landslides of any depth, but relative effectiveness decreases with increasing depth of movement.
	Shallow (0.5 to 3 m)	8	
	Medium (3 to 8 m)	6	
	Deep (8 to 15 m)	2	
	Very deep (> 15 m)	0	
Rate of movement (Varnes, 1978)	Moderately to fast	2	Seeding can be appplied remotely, by helicopter if necessary. However, it needs time to become established (especially trees) and this may limit application in moderately to fast movements.
	Slow	6	
	Very slow	8	
	Extremely slow	8	
Groundwater	Artesian	8	Applicable irrespective of groundwater conditions. Effects on groundwater levels only indirect through reduced infiltration and suctions. Potential difficulties and/or extra maintenance required where groundwater is low or absent.
	High	8	
	Low	6	
	Absent	6	
Surface water	Rain	8	Water courses should be diverted. Even small localized flows may hinder establishment.
	Snowmelt	8	
	Localized	6	
	Stream	4	
	Torrent	0	
	River	0	
Maturity		6	Apparently simple and long practiced technique, it requires careful selection of species. Ongoing discussion about real benefits and limits of applicability.
Reliability		6	Effects on stability only indirect and difficult to quantify.
Implementation		8	Easily implemented with widely available equipment. However, it requires intense maintenance during early stages, say up to 3 years in certain cases.
Typical Cost		10	Low, where applicable.

Note
Ratings are given on a scale of 1 to 10; the higher the grade, the most suitable is the specific method under consideration to use in landslides of the given characteristics, evaluated individually. Overall suitability to specific case under consideration may be obtained by a weighted average of these ratings, with user defined weights. Zero rating means "not applicable"

MITIGATION THROUGH REDUCTION OF HAZARD

3 MODIFYING THE SURFACE WATER REGIME – SURFACE DRAINAGE

3. 5 VEGETATION – HYDROLOGICAL EFFECTS

References:

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MITIGATION THROUGH REDUCTION OF HAZARD

3 MODIFYING THE SURFACE WATER REGIME – SURFACE DRAINAGE

3.6 HYDRAULIC CONTROL WORKS (CHANNEL LINING AND CHECK DAMS)

Description

Two types of hydraulic control works are normally used: stream channel linings and check dams (Figure 1).

Channel linings are effective both in maintaining channel alignement and in reducing the frequency and volume of debris flows. They are most effective if applied over the entire reach of an unstable channel.

Channel linings are usually made by masonry or stone pitching with high-quality concrete, preferably reinforced by steel fiber to resist abrasion; protruding boulder are set in the concrete to dissipate the energy of waterflow. Where required, boulders may be tied together by C chaped steel bars drilled and grourted ito adjacent boulders.

Check dams are small sediment-storage dams built across the channels of steep gullies to slow down the flow, dissipating part of the energy, to stabilize the channel bed, thus preventing or mitigating landslides caused by basal erosion. They are also used to control the frequency and volume of channelized debris-flows and/or to control ravelling and shallow slides in the source area of debris-slides. Channelized debris flows are associated with channel gradients over 25° and obtain most of their volume by scouring the channel bed.

Check dams serve three purposes when installed in the channels (Chatwin et al., 1994):

1. To mitigate the incidence of failure by reducing the channel gradient in the upper channel;
2. To reduce the volume of channel-stored material by preventing down cutting of the channel with subsequent gully sidewall destabilization and by providing toe support to the gully slopes;
3. To store debris-flow sediment when installed in the lower part of the channel.

Check dams can be constructed of timber cribs (Figure 2) or concrete cribs, concrete mortared rock or plain or stone faced reinforced concrete (Figure 3). Concrete mortared rock dams do not usually exceeed 8 m in height, whereas concrete or timber crib dams do not exceed 2 m.

The spacing of check dams along the channel depends on the natural and infill gradient of the channel infill and the dam height; as an example, a 2 m high dam in a 20° degree channel with 10° sloping channel infill will be spaced every 12 m. (Highlands and Bobrowsky, 2008).

Reference may be made to Popescu M.E., Sasahara K. (2009) for further discussion and examples of check dams for the mitigation of debris flows.

Channel linings are usually less expensive than check dams, especially if a long reach is to be stabilized; check dams are preferable, however, if the banks are very unstable because a dam can be keyed into the bank, providing toe support, enhancing stability. Check dams are expensive to contruct and therefore are usually built only where necessary to protect vulnerable elements downstream.

Design

Channel linings need to be designed to have adequate stability against disturbance by the current; current velocity and bank slope angle govern the minimum and median block size in rip rap and stone linings. The local stability of the lining will also need to be verified with respect to static equilibrium under various groundwater conditions. Where concrete slabs or equivalent systems are used, special care will need to be paid to relieving water pressures at the contact with the underlying soil, especially where the lining obstructs free drainage towards the channel.

Lateral stream erosion and scour by spillway water are the main drawbacks of check dams. To prevent check dam failure the following recommendations apply:

- During construction the wingwalls must be tied into the gully walls and the streambed to withstand backfill pressures and lateral scour; wingwalls should slope about 70% and extend a minimum of 1÷2 m into the banks;
- The foundation of the dam should have a minimum width of 1/3 the total height of the dam and be deeper than any scour holes likely to develop;
- Downstream aprons (Figure 3b) or stilling basins should be provided, where feasible;
- The dynamic equilibrium of the whole reach should be considered, remembering that sediment accumulated by check dams tends to be replaced by increased streambed erosion downstream.
- Backfilling the dam, rather than allowing it to fill naturally, reduces the dynamic loading on the structure and results in a more stable design. The slope of the backfill should be less than 1/2 the channel gradient.

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Figure 1: Pitched stone channel lining and concrete check dams, Gimillan, nr. Cogne (AO), Italy (photo: G. Vaciago, SGI-MI)

Figure 2: Timber crib wall check dams, Trafoi, Italy
(source: Highlands and Bobrowsky, 2008)

Figure 3: Selective stone faced concrete check dam and debris accumulation basin, Gimillan, nr. Cogne (AO), Italy. (a) downstream and (b) upstream view
(photo: G. Vaciago, SGI-MI)

(a)

(b)

MITIGATION THROUGH REDUCTION OF HAZARD			
3 MODIFYING THE SURFACE WATER REGIME – SURFACE DRAINAGE			
3.6 HYDRAULIC CONTROL WORKS			
APPLICABILITY			
Class	**Descriptor**	**Rating**	**Notes**
Type of movement (Cruden & Varnes, 1996)	Falls	0	Most suited to rotational or pseudo-rotational slides; may be useful to reduce toppling hazard in certain conditions.
	Topples	0	
	Slides	8	
	Spreads	0	
	Flows	8	
Material	Earth	8	Mainly applicable to landsliding involving earth and debris. Applicability in rock limited by typical slope geometry and failure mode.
	Debris	8	
	Rock	0	
Depth of movement	Superficial (< 0.5 m)	8	Typically applicable to relatively small and/or shallow landslides. The implications of large scale filling and procurement typically make this technique impractical for deep and very deep slides. On the other hand, it may be the only suitable technique in very large landslides, besides drainage.
	Shallow (0.5 to 3 m)	8	
	Medium (3 to 8 m)	8	
	Deep (8 to 15 m)	6	
	Very deep (> 15 m)	4	
Rate of movement (Varnes, 1978)	Moderately to fast	0	Can be carried out only when the rate of movement is extremely slow or at most very slow (maximum 1.5 m/year).
	Slow	0	
	Very slow	6	
	Extremely slow	8	
Groundwater	Artesian	6	Applicable in all groundwater conditions. Adequate drainage must be provided at the back of impervious linings, especially where artesian or high ground water levels exist.
	High	6	
	Low	8	
	Absent	8	
Surface water	Rain	6	Applicable to water courses. Most useful in high energy environments. Unaffected by and ineffectual with respect to rain and snowmelt.
	Snowmelt	6	
	Localized	8	
	Stream	8	
	Torrent	10	
	River	8	
Maturity		8	Well established technique. Potential benefits and limits of applicability are well understood.
Reliability		8	The reliability of the technique depends on the reliability of the evaluation of the demand in terms of hydraulic and/or debris flows.
Implementation		6	May be complex in permanent water courses. Requires heavy construction equipment which may have access restrictions.
Typical Cost		5	Moderate to high, depending on access conditions and availability of materials.

Note
Ratings are given on a scale of 1 to 10; the higher the grade, the most suitable is the specific method under consideration to use in landslides of the given characteristics, evaluated individually. Overall suitability to specific case under consideration may be obtained by a weighted average of these ratings, with user defined weights. Zero rating means "not applicable"

MITIGATION THROUGH REDUCTION OF HAZARD	
3 MODIFYING THE SURFACE WATER REGIME – SURFACE DRAINAGE	
3.6 HYDRAULIC CONTROL WORKS	
References: Chatwin S.C., Howes D.E., Schwab J.W., Swanston D.N. (1994). "A guide for menagement of landslide-prone terrain in the Pacific Northwest" 2nd edition, Ministry of Forests, British Columbia, 200 p. http://www.for.gov.bc.ca/HFD/Pubs/Docs/Lmh/Lmh18.htm Highland L.M., Bobrowski P. (2008). "The Landslide Handbook – A Guide to Understanding Landslides". Circular 1325, U.S. Geological Survey. Popescu M.E., Sasahara K. (2009). "Engineering measures for Landslide disaster mitigation". In: Landslide Risk reduction", K. Sassa, P.Canuti (eds.), Spinger-Verlag, Berlin, 609-631.	

MITIGATION THROUGH REDUCTION OF HAZARD

3 MODIFYING THE SURFACE WATER REGIME – SURFACE DRAINAGE

3.7 DIVERSION CHANNELS

Description

Diversion channels are mostly artificial channels designed to divert excess amount of water to prevent flooding, erosion and landsliding. On the basis of the purpose of use, diversion channels can be grouped into:

- river diversion channels;
- runoff diversion channels.

Runoff diversion channels are describe in fact sheet 3.1. River diversion channels are discussed here.

River diversion channels are artificial channels built or used to divert all or part of the river flow from the toe of a slope/landslide, either to prevent or remediate toe erosion, or to make space for the implementation of other mitigation measures, as was carried out for example on the Taren landslide (Figure 1, Kelly and Martin, 1985); it may be temporary or permanent based on the duration of use.

Diversion channels, often in tunnel, are also used to divert water from landslide dams to protect the areas below and around the landslides dams; they are used either after the event, as for the Val Pola, Italy 1987 landslide and for the landslides reported in Table 1, or as a preventive measure, as carried out for the Séchilienne Landslide in France (Durville et al., 2004).

Table 1: Examples of landslide dam break and flood prevention through diversion channel (Schuster, 2006; Liu et al., 2010)

N°	Case/year	Problem	Material in dam	Mitigation	Consequences
1	Madison River, Montana, USA/1959	$21x10^6$ m^3 landslide, triggered by earthquake, created 70m high dam	Rocks, gravels	75 m wide open channel spillway designed for a discharge of 280 m^3/s	Prevented dam failure
2	Pisque River, Northern Ecuador/1990	$3.6x10^6$ m^3 landslide, triggered by irrigation wastewater, created 58m high dam	Silty sands from volcanic tuff, fragments and blocks of soft tuff, sandstone, breccia	100m x11m x 9m open channel constructed in 7 days to reduce the severity of expected flood by limiting the depth of the lake	Dam failed due to erosion at channel, but reduced 40% of the lake discharge
3	Yingong River, Eastern Tibet, China/2000	$300x10^6$ m^3 debris avalanche dammed the river; the dam was 60 to 100m high, 2.5 km long, 2.5 m wide	-	Open channel spillway	Dam failed by overtopping and eroding the discharge channel; severe property and life loss downstream
4	Tongkpo River, Sichuan, China	Earthquake triggered landslide, creating Tanjiashan barrier lake with storage capacity of $3.2x10^8$ m^3	Sandstone	Discharge channel 890 m long, 13 m deep and 8 m wide	The lake water was drained, reducing the risk of flooding upon dam breakage

Landslides dams (Figure 2) cause mainly two types of floods: 1) upstream flooding as in the impoundment fills (Figure 3); or 2) downstream flooding resulting from failure of the dam (Figure 4). A landslide dam and its impounded lake may last from a few hours to thousand of years, depending on:

- Rate of inflow to the lake, which is based on the size of the drainage basin upstream of the dam and on the amount and rate of precipitation into the basin.
- Size and shape of the dam. High dams will need a longer time to fill than low dams and wide dams will be more resistant than narrow dams to failure upon overtopping.
- Rate of seepage through the dam.
- Resistance to erosion at the dam surface and subsurface.

Figure 1: River diverted to allow construction of toe berm, Taren landslide, UK (source: Kelly and Martin, 1985)

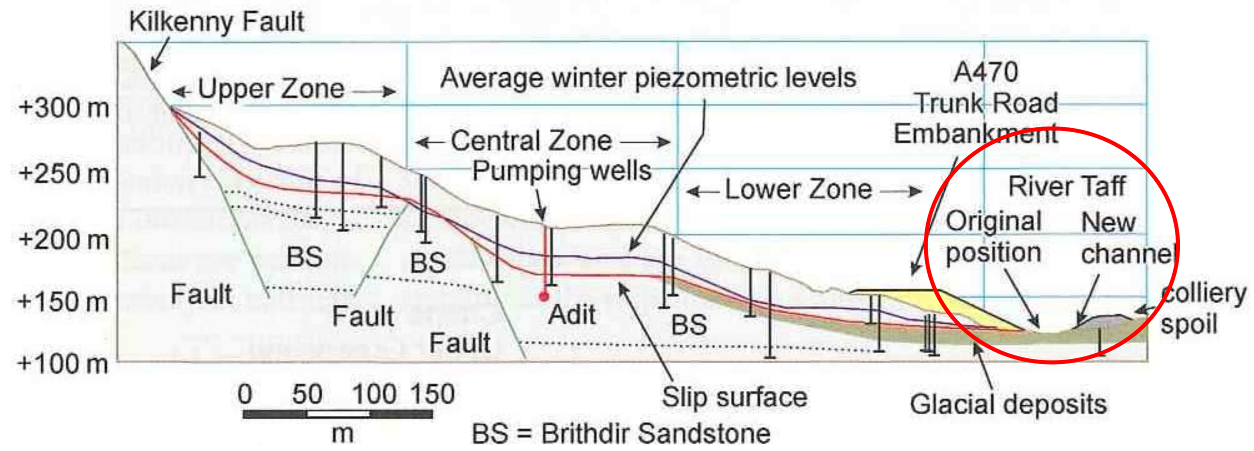

Figure 2: Mechanism and consequences of landslide dams (source: www.kingston.ac.uk)

Figure 3: Flooding induced by landslide dam, Randa, Switzerland, 9 May 1991 (source: www.crealp.ch)

MITIGATION THROUGH REDUCTION OF HAZARD

3 MODIFYING THE SURFACE WATER REGIME – SURFACE DRAINAGE

3.7 DIVERSION CHANNELS

Istantaneous lake depletion can be caused by rapid erosion of the landslide dam on overtopping or dam collapse caused by piping and internal erosion.
Significant technical and financial resources are necessary to design and construct diversion channels for channelling the huge volumes of water involved. Diversion channels are expensive and they take a long time to build.
The cases presented in Table 1 show the difficulty of dealing with landslide dams. In cases 2 and 3 the discharged channels constructed across the landslide dam were not successful because of retrogressive erosion of the channels. In cases 1 and 4 the surface geology was mainly composed of weathered rock materials and erosion of these channels was minimal, so they were successful. In order to minimize the risk of erosion, the diversion channel should be designed and constructed with all the necessary precautions typical of major hydraulic works. However, this is seldom possible in an emergency. To minimize this risk, significant temporary pumping was carried out at the Val Pola landslide dam to allow sufficient time to construct erosion protection works in the emergency spillway channel. A more radical solution is to place the diversion channel in tunnel, but this requires a much longer time and can hardly be considered in an emergency.

Design

Diversion channels are complex hydraulic structures that need to be designed accordingly. Critical aspects are the design of headworks and outlet, cross section, horizontal and vertical alignment, flow speed and profile, bank stability, lining of banks and base. All design calculations are based on design flows derived from full hydrological analysis of the catchment area.

To design and construct a diversion channel across a landslide dam, it is necessary to estimate the amount of discharge from the dam. The accuracy of dam-break flood routing is affected by many hydrological and topographical factors; the calculated results may be quite different from the real situation. An example of a dam breaking flood analysis is represented by case 4 of Table 1; the equations used to calculate the dam-break flood are summarized below :

- The maximum flood discharge at the entrance (Qmax) has been calculated according to the formula of broad-crest weirs:

  $$Q_{\max} = \delta \cdot m \cdot b \cdot 2 \cdot g \cdot H_o$$

  where: b = width of the weir; H_o = effective water head during the maximum flood; δ = coefficient of lateral contraction; m = coefficient of discharge, g = acceleration of gravity.
- The maximum flooding discharge at a distance L from the lower reaches of the landslide dam (Q, in m^3/s) has been calculated on the basis of the following equation:

  $$Q = \frac{W}{\dfrac{W}{Q_{\max}} + \dfrac{L}{v_{\max} \cdot k}}$$

  where: L = distance downstream the landslide dam in meters; W = total storage capacity of the reservoir in m^3; V_{max} = velocity of the maximum flood discharge im m/d; K = empirical coefficient (1.1÷1.5 in mountain areas; 1 in hilly areas; 0.8÷0.9 in plain areas).

Figure 4: Flooding induced by breeching of landslide dam
(source: http://yeehowcentral.blogspot.com/2008_06_01_archive.html Posted by Dr. Jerque)

MITIGATION THROUGH REDUCTION OF HAZARD			
3 MODIFYING THE SURFACE WATER REGIME – SURFACE DRAINAGE			
3.7 DIVERSION CHANNELS			
APPLICABILITY			
Class	**Descriptor**	**Rating**	**Notes**
Type of movement (Cruden & Varnes, 1996)	Falls	6	Appropriate for any type of landslide, in so far as it may form a landslide dam. Diversion to prevent toe and/or basal erosion typically relevant to all types of slides.
	Topples	6	
	Slides	8	
	Spreads	6	
	Flows	6	
Material	Earth	8	Appropriate for landslide in any type of material, in so far as it may form a landslide dam.
	Debris	8	
	Rock	8	
Depth of movement	Superficial (< 0.5 m)	0	Typically applicable and justified only to very deep landslides.
	Shallow (0.5 to 3 m)	0	
	Medium (3 to 8 m)	4	
	Deep (8 to 15 m)	6	
	Very deep (> 15 m)	10	
Rate of movement (Varnes, 1978)	Moderately to fast	8	The works are carried out outside the landslide body. However, they may be located in the run-out area of larger slides than considered in design, thus special care is required in areas susceptible to run out of fast to very fast landslides.
	Slow	8	
	Very slow	8	
	Extremely slow	8	
Groundwater	Artesian	8	Applicable to all landslide groundwater conditions. Adequate drainage must provided at the interface between impervious channel linings and natural soil.
	High	8	
	Low	8	
	Absent	8	
Surface water	Rain	6	Applicable to water courses. Most useful in high energy environments. Unaffected by and ineffectual with respect to rain and snowmelt.
	Snowmelt	6	
	Localized	6	
	Stream	8	
	Torrent	8	
	River	8	
Maturity		6	Simple technique. Potential benefits and limits of applicability are well established.
Reliability		6	In emergency works, the reliability depends on the possibility of implementing appropriate erosion control. Otherwise depends on the hazard study.
Implementation		6	Major erthworks or even tunneling works. Time consuming. Compex to implement in emergencies.
Typical Cost		2	Very high. Only justified for major risk situations.

Note
Ratings are given on a scale of 1 to 10; the higher the grade, the most suitable is the specific method under consideration to use in landslides of the given characteristics, evaluated individually. Overall suitability to specific case under consideration may be obtained by a weighted average of these ratings, with user defined weights. Zero rating means "not applicable"

MITIGATION THROUGH REDUCTION OF HAZARD

3 MODIFYING THE SURFACE WATER REGIME – SURFACE DRAINAGE

3.7 DIVERSION CHANNELS

References:

Durville J.L., Effendiantz L., Pothérat P., Marchesini P. (2004). "The Séchilienne landslide". In: Identification and mitigation of large landslide risk in Europe. Advances in risk assessment. Ch. Bonnard, F. Forlati, C. Scavia (eds.), European commission Fifth Framework Programme, IMIRILAND Project, A.A. Balkema, Leiden, 251-269.

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MITIGATION THROUGH REDUCTION OF HAZARD

FACT SHEET 4

MODIFYING THE GROUNDWATER REGIME – DEEP DRAINAGE

MITIGATION THROUGH REDUCTION OF HAZARD

4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE

4.0 GENERAL

Description

In saturated soils, drainage is often the best remedial measure against slope instability due to the important role played by pore-water pressure in reducing the shear strength of the soil. Because of its high stabilization efficiency in relation to cost, drainage of groundwater is widely used and is generally the most successful stabilization method. Moreover, drainage proves suitable for a large number of cases, even when the landslide is very deep and structural measures are not effective (Popescu, 2002).

Approach to design

The mechanism of drainage inside slopes involves a decrease in pore pressures in the subsoil and consequently an increase in effective stresses and soil shear strength in the whole drained domain. In particular, the increase in soil shear strength along the potential sliding surface of the landslide body, due to working of drains, is responsible for the slope stability improvement. Therefore the first step in the design of a drainage system is to determine the pore pressure change that is necessary to increase the factor of safety of the slope to the required value. The next step is to design the geometric configuration of drains that will result in the required pore pressure change. The effect of the drainage system is usually analyzed for the steady-state condition, which is attained some time after drainage construction (i.e. in the long term) (Urciuoli, 2008).

The steady-state condition is usually analyzed by assuming continuous infiltration of water at the ground surface to recharge the water table. In the literature, results of steady-state analyses are often presented in non-dimensional design charts that practitioners generally use to design drainage systems.

After drain installation, a transient phase of equalization of pore pressures occurs and two aspects have to be evaluated in the design referring to this phase (Urciuoli, 2008):

a. whether the delay until the drains are completely effective is affordable,
b. Whether settlements associated with de-watering will damage buildings and infrastructures at the ground surface.

During the phase of construction and of working as well, it is important to evaluate and to check the conditions of drains by means of piezometers. Indeed, pore pressure changes are the most direct and useful indicators of drains being in good working condition. Measurements of surface and deep displacements are good indicators of overall slope stability.

Drain types

The main deep drainage systems include:

- Shallow trenches filled with free-draining material (Fig. 1);
- Deep trenches filled with free-draining material;
- Sub-horizontal drains (conventional drilling) (Fig. 2);
- Sub-horizontal drains (directional drilling);
- Vertical small diameter (< 800mm) wells - relief of artesian pressure;
- Vertical small diameter (< 800mm) wells - under drainage of perched aquifer;
- Vertical small diameter (< 800mm) wells - pumps;
- Vertical large diameter (> 1500mm) wells - gravity drainage through base conductor (Fig. 3);
- Caisson (> 5 - 6 m), with gravity drainage (and secondary sub horizontal drains) (Fig. 4);
- Drainage tunnels, adits, galleries, with secondary drains or as outlet for wells (Fig. 5).

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Figure 1: Shallow trenches

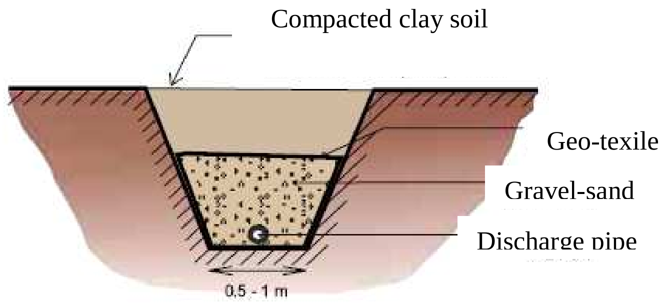


Figure 2: Sub-horizontal drains

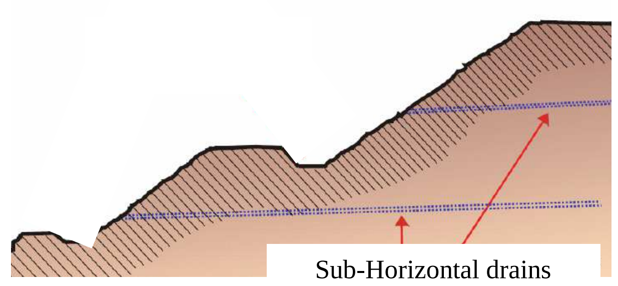


Figure 3: Vertical large diameter wells

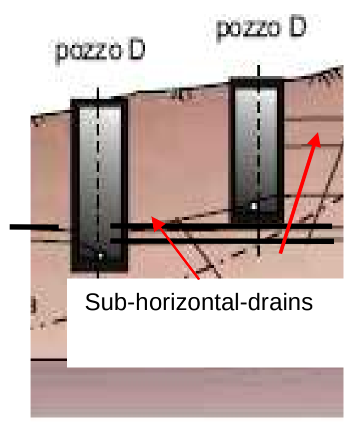


Figure 4: Well with gravity drainage and secondary drains

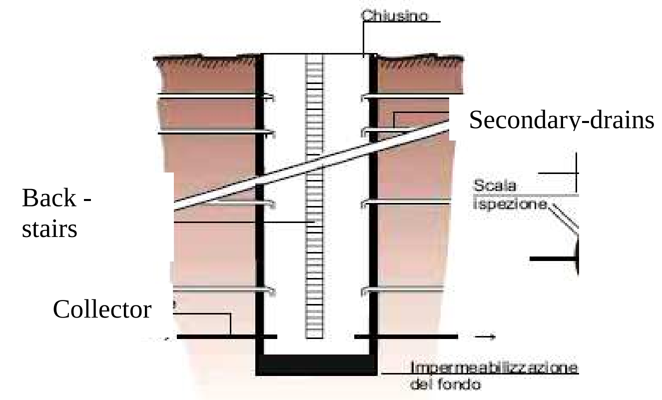
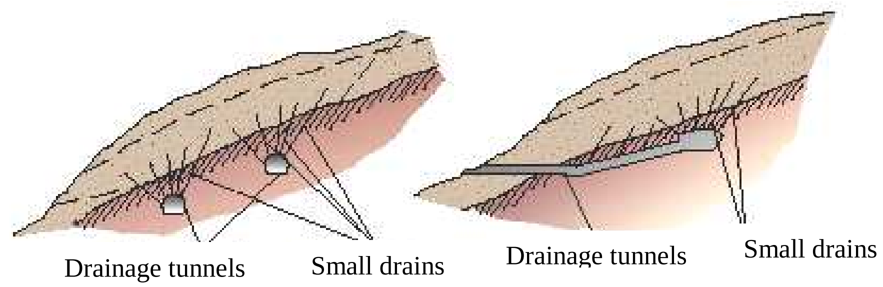


Figure 5: Galleries



MITIGATION THROUGH REDUCTION OF HAZARD

4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE

4.1 SHALLOW TRENCHES FILLED WITH FREE-DRAINING MATERIAL

Description

Drain trenches are used to stabilize translational or rotational slides which occur typically in highly weathered fine-grained soils, characterized by permeability higher than that of the underlying layer. Tipical layouts of shallow trenches, with main and possibly secondary branches, and typical cross sections are shown in Figs. 1a, b, c, d (Urciuoli, 2008).

Trenches should be excavated deep enough to intercept the regions of positive pore pressures. Shallow trenches can be excavated by means of an excavator up to a depth of approximately 5 m from the ground surface (Fig. 2). The width of the trench is dependent on the type of excavator being used and may vary from 0.5 to 1.0 m. In open areas, trenches can have sloping sides, the gradient of which is based on stability consideration (Fig. 3). Where there is not enough space, trench sides have to be formed to vertical and should be properly supported (Fig. 4). Guidelines on the design of lateral support to excavation are given in many publications, e.g. BS 6031:1981 (BSI 1981). However problems of trench instability can be reduced by opening up trenches in short lengths and backfilling the trench within a short time after excavation. (Urciuoli, 2008). Trenches need to have a high discharge capacity to avoid the saturation of the backfilling material or of the lower portion of it. This can be achieved providing a drainage layer of gravel materials or installing at the bottom of the trench a perforated pipe (with slots on the upper part). The perforated pipe should be wrapped with a geotextile to prevent the clogging of the slots by fine soil particles (Fig. 5). A compacted clay cover should be placed on the top of the trench to prevent ingress of surface water, which should be drained by means of a system of surface drainage network. The impermeable cover should have a minimum thickness of 0.5 m and should be compacted in layers (Fig. 5).

Trenches should be constructed starting from the lowest point in the area to be drained, so that they can drain water during construction. Inspection wells that intercept the trenches should be installed to allow:

- monitoring of the working condition of the drainage system, possibly by measuring the flow;
- maintenance, possibly flushing of the perforated pipe.

The reduction of pore water pressure varies along the slope longitudinal section, the maximum decreasing occurs at a distance from the head trench equal to 3-4 times the space along the cross section. Therefore the length of trenches is usually extended 3-4 time the wheelbase outside the slide area.

Design

The first step in the design of a drainage system is the determination of the pore pressure change that is required to increase the factor of safety of the slope to the design value. The next step is to design the geometric configuration of drains that will result in the required pore pressure change.

The design of drain trenches can be carried out by using numerical analyses or easily by adopting design charts.

In the first case drainage works is analysed by means of numerical codes (DEM or FEM) and the problem may be solved by taking soil stratigraphy and heterogeneity into account and by assuming climate conditions acting at the upper boundary. The pore pressures calculated along the critical sliding surface should be used in slope stability analysis.

In the second case, non-dimensional charts obtained for homogeneous soil and very simple geometric schemes are used to estimate pore pressure, lowered by drains. Design charts are a general tool: they cannot consider hydraulic conditions at ground surface according to a seasonal trend, which necessarily depends on typical climatic features of the region being considered.

In fact methods of analyzing the stabilization effect of drain trenches commonly available in the literature (e.g., Hutchinson 1977, Desideri et al. 1997) model the groundwater regime as a steady-state phenomenon (seepage), and assume the presence of a film of water at the ground surface. In areas where the weather is not very rainy, such as in southern Europe, this assumption underestimates the effects of drains on slope stability (Urciuoli, 2008).

The majority of design charts are used to obtain the geometric configuration from the global efficiency of the drainage system, determined as a function of the pore water distribution that guarantees the safety factor chosen by the designer.

The design charts proposed by Urciuoli (2008) are based on steady-state analysis carried out for drains operating in 3D conditions, assuming a film of water fixed at ground surface. For more details about the boundary condition and the domain analysed see D'Acunto & Urciuoli, 2006. The results pointed out are that the lowering of the water table caused by drains is not homogeneous with depth in the drained domain: it depends upon the distance of the examined point from the drain boundaries and especially from the ground surface.

Figure 1: Shallow trenches with only main branches: a) Plan, b) Cross section. Shallow trenches, with main and secondary branches: c) Plan, d) Longitudinal section (from Urciuoli, 2008).

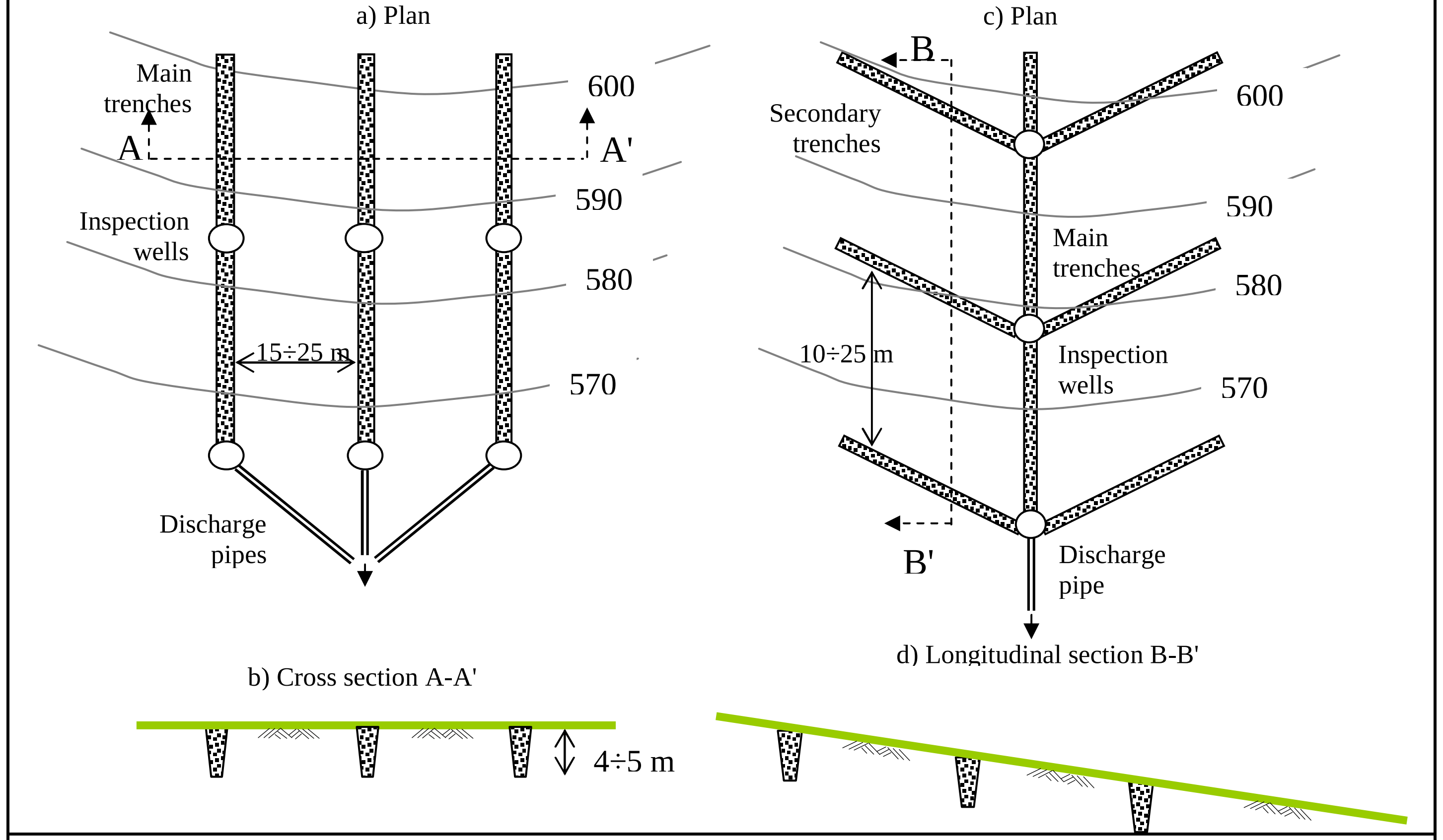
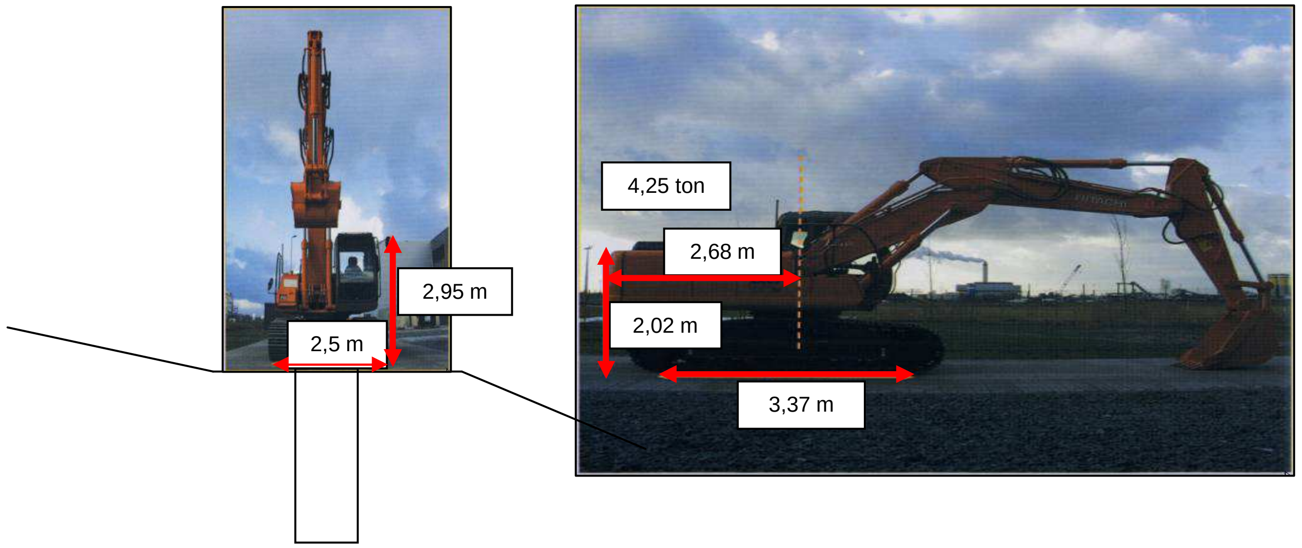


Figure 2: Excavator used for trenches up to a depth of approximately 5 m from the ground surface



MITIGATION THROUGH REDUCTION OF HAZARD

4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE

4.1 SHALLOW TRENCHES FILLED WITH FREE-DRAINING MATERIAL

The drainage effect is weaker in the deepest zone of the slope. Because a simplification is required to handle the problem more manageably and to make the design charts, the model of infinite slope (1D) is adopted. According to that, the Author schematized the 3D pore pressure distribution resulting from the action of drains as a 1D distribution (equivalent to 3D distribution as regards its influence on slope stability). Therefore the effect of drainage is evaluated by means of the average efficiency $\overline{E}$ along the sliding surface Γ, which expresses the difference between the initial and current value of mean pore pressure (at a generic time *t*), normalized to the initial value:

$$\overline{E}(t,\Gamma)=\frac{\overline{u}(0,\Gamma)-\overline{u}(t,\Gamma)}{\overline{u}(0,\Gamma)}$$

Finally, for the steady-state solution (attained at long term), the function $\overline{E}_{\infty}$ can be used:

$$\overline{E}_{\infty}(\Gamma)=\frac{\overline{u}(0,\Gamma)-\overline{u}(\infty,\Gamma)}{\overline{u}(0,\Gamma)}$$

The function $\overline{E}_{\infty}$ plays a key role in designing slope stabilization by drains, because it considers the final distribution of pore pressure $[\overline{u}(\infty,\Gamma)]$, used in the calculation to obtain the desired improvement in slope stability; the effectiveness of drains is correctly analyzed by considering the groundwater regime as a steady-state phenomenon.
In practice, $\overline{E}_{\infty}(\Gamma)$ is calculated, after determining $\overline{u}(\infty,\Gamma)$ from slope stability analysis, as the pore pressure that guarantees the safety factor chosen by the designer. From $\overline{E}_{\infty}(\Gamma)$, by means of non-dimensional charts, the designer can determine the geometric characteristics of the drain system.
By using the pore water pressure distribution obtained by numerical analysis and adapting them to equivalent 1-D domain, the value of $\overline{E}_{\infty}(D)$ has been calculated for trenches with secondary branches and represented in design charts as a function of the following parameters:

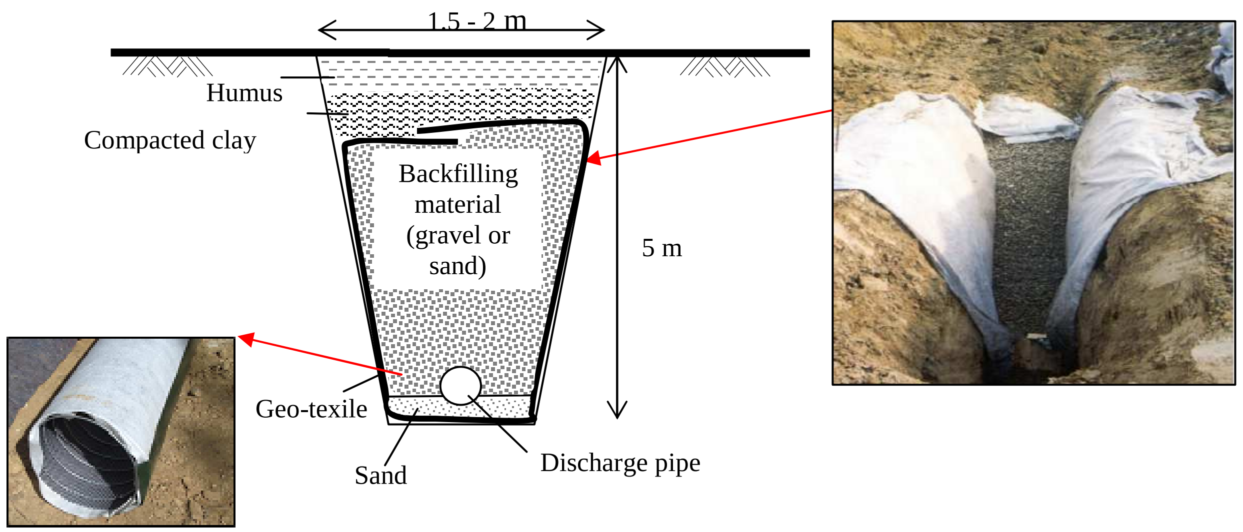
H = depth of analysed volume Ω,
H_0 = depth of drain,
D = depth of the plane on which efficiency is evaluated (correspondent to sliding surface),
L_y = longitudinal length of the analysed volume Ω (in the case of trenches it is the spacing between principal branches of drain trenches),
S = spacing between secondary branches of drain trenches,
i = spacing between horizontal drains,
l_2 = length of secondary branches of drain trenches,
$l_1 = L_y - l_2$.

Four design charts, one for each plane on which the efficiency is evaluated, are reported below.

Figure 3: Excavation of trenches with sloping sides

Figure 4: Excavation with vertical sides and supports

Figure 5: Scheme of a shallow trench (from Urciuoli, 2008)



MITIGATION THROUGH REDUCTION OF HAZARD

4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE

4.1 SHALLOW TRENCHES FILLED WITH FREE-DRAINING MATERIAL

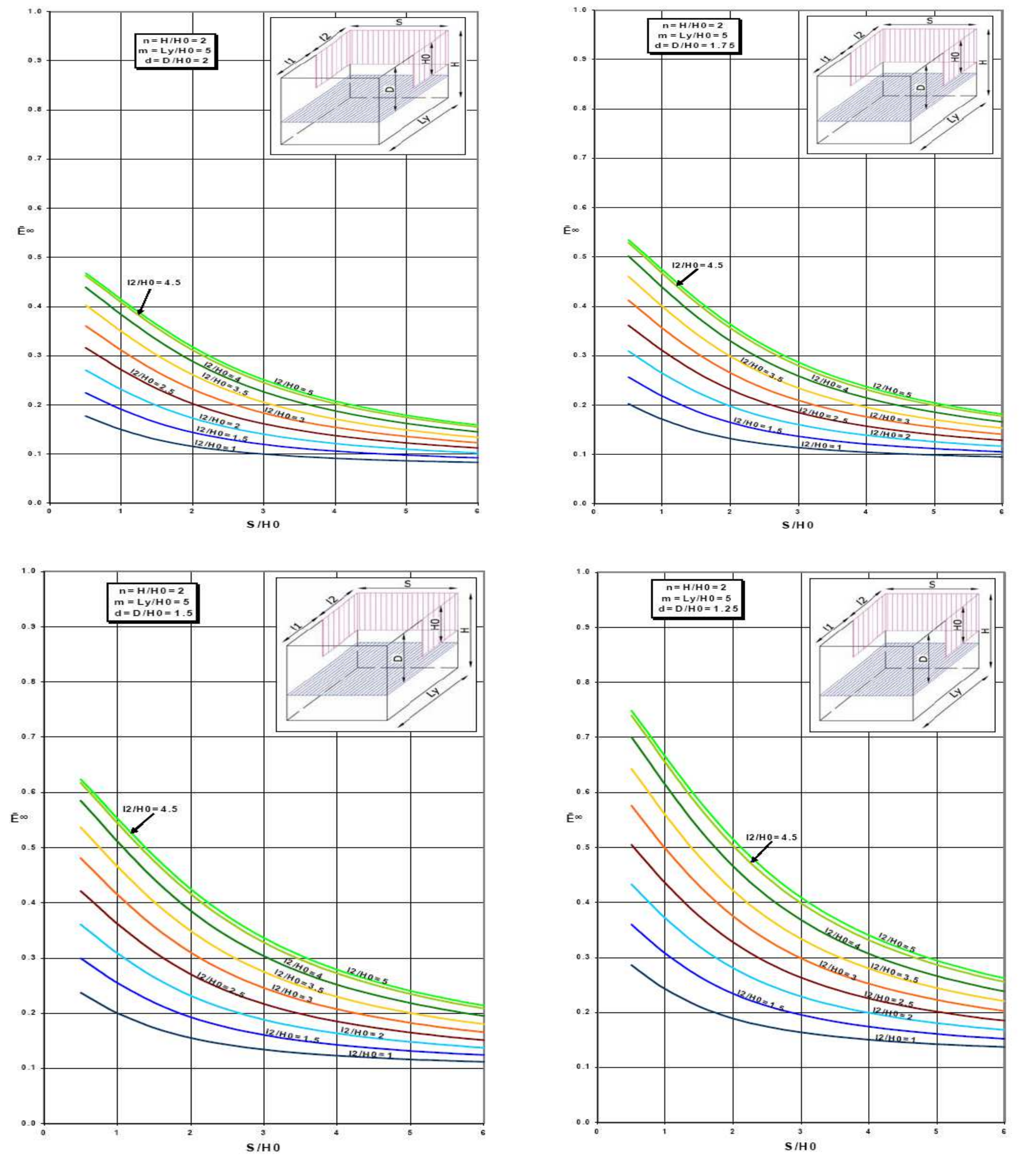


Figure 6: Examples of shallow trenches. The upper part of the system is covered by stones in order to lower the environmental impact

Figure 7: Phases of the construction of a drainage trench using geocomposites (assembly of components).

MITIGATION THROUGH REDUCTION OF HAZARD			
4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE			
4.1 SHALLOW TRENCHES FILLED WITH FREE-DRAINING MATERIAL			
APPLICABILITY			
Class	**Descriptor**	**Rating**	**Notes**
Type of movement (Cruden & Varnes, 1996)	Falls	0	Drain trenches are often used to stabilize shallow translational slides of large extension.
	Topples	0	
	Slides	6	
	Spreads	0	
	Flows	4	
Material	Earth	8	Translational slides occur typically in fine-grained soils strongly altered and characterized by permeability much higher than that of the underlying layer.
	Debris	6	
	Rock	0	
Depth of movement	Superficial (< 0.5 m)	8	The maximum depth for the shallow drainage system is 5-6 m therefore the best efficiency value is calculated at a depth equal or less than 5-6 m. As a consequence shallow drain trenches are suitable when the depth of slip surface is not deeper than 5-6 m.
	Shallow (0.5 to 3 m)	8	
	Medium (3 to 8 m)	4	
	Deep (8 to 15 m)	0	
	Very deep (> 15 m)	0	
Rate of movement (Varnes, 1978)	Moderately to fast	0	The steady-state condition is attained some time after drainage construction (i.e. in the long term) in fact after drain installation, a transient phenomenon of equalization of pore pressures occurs. Drains are completely effective after a delay; therefore they represent a suitable mitigation method for very slow landslides.
	Slow	8	
	Very slow	8	
	Extremely slow	8	
Groundwater	Artesian	2	This system is suitable for shallow freatic water- table.
	High	6	
	Low	2	
	Absent	0	
Surface water	Rain	8	The methods of analyzing the stabilization effect of drains commonly available in the literature assume the presence of a film of water at the ground surface. However in areas where the weather is not very rainy, such as in southern Europe, this assumption underestimates the effects of drains on slope stability. The seasonal variation of rain-infiltration may be taken into account, as they influence the system performance.
	Snowmelt	8	
	Localized	0	
	Stream	0	
	Torrent	0	
	River	0	
Maturity		8	Technique and design process are well established and widely used in suitable conditions.
Reliability		7	The good working depends strongly on the maintenance, possibly by flushing the perforated pipe. However the life-service is long enough.
Implementation		7	Technologies used for excavation are well-kown and long-established and uncertainties are low.
Typical Cost		7	Lless costly than other types of stabilization works and suitable for a large number of cases, even when structural measures are not effective.

Note
Ratings are given on a scale of 1 to 10; the higher the grade, the most suitable is the specific method under consideration to use in landslides of the given characteristics, evaluated individually. Overall suitability to specific case under consideration may be obtained by a weighted average of these ratings, with user defined weights. Zero rating means "not applicable"

MITIGATION THROUGH REDUCTION OF HAZARD

4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE

4.1 SHALLOW TRENCHES FILLED WITH FREE-DRAINING MATERIAL

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MITIGATION THROUGH REDUCTION OF HAZARD

4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE

4.2 DEEP TRENCHES FILLED WITH FREE-DRAINING MATERIAL

Description

The schematic map of deep trenches, with main and possibly secondary branches, and typical cross sections are shown in Figs. 1a,b,c,d (Pun & Urciuoli, 2008). Essentially only the technology to excavate the trench is different from that used for shallow trenches. Deep trenches can reach the maximum depth of 30 m and are excavated by means of grab shells (Fig. 2). The sides of the trenches, being vertical, should be supported by means of slurry, e.g. polymeric mud (Fig. 3), therefore costs increase very much respect to shallow trenches.
Because of the high depth, it is difficult to provide the good laying of the discharge pipe; moreover the volume of materials necessary to fill trenches is very large. Therefore new technologies are continuously advancing, for example drainage cage may be dropped directly inside the trenches. Two new types of technologies adopted conveniently for deep trenches are described below:

- Narrow trench fitted with a draining geocomposite with a high capacity collection surface, buried with slightly compacted excavated soil. This is a geocomposite consisting of a draining core combined with two geotextile filters with a socket at the base for fitting the drainage tube. The features of this system are: excellent filtering, constant hydraulic efficiency, good excavation volume and no soil to dispose of, total or drastic reduction of inert materials, higher output and extra safety in the yard. All these features make draining with this system an innovative technique compared to traditional systems. Vertically continuous draining is possible for deep trenches by combining this system with suitable draining composites by securing them and superimposing them by means of suitable, simple measures (Figs. 3a, b).
- Deep trenches can be carried out as panels constituted by ''aerated concrete'': gravel with high permeability (10^{-1} m/s) and cement with a good compression strength (Fig. 4). The technology used is that used for diaphragms, therefore any depth can be reached. The panels usually have the plan dimensions: 0,8-1m x 2,5-3m; first the odd-numbered ones are constructed. This system characterized by 'aerated concrete' can be realized as secant piles as well (Fig. 4), but the previous technique is faster.

About the maintenance and monitoring, the same consideration for the shallow drain system can be made.

Design

The design criteria of deep trenches are the same as adopted for the shallow trenches. Numerical analyses can be carried out or, more easily, design charts can be used (see fact sheet 4.1).

Figure 1: Deep trenches with only main branches: a) Plan, b) Cross section. Deep trenches, with main and secondary branches: c) Plan, d) Longitudinal section.(from Urciuoli, 2008)

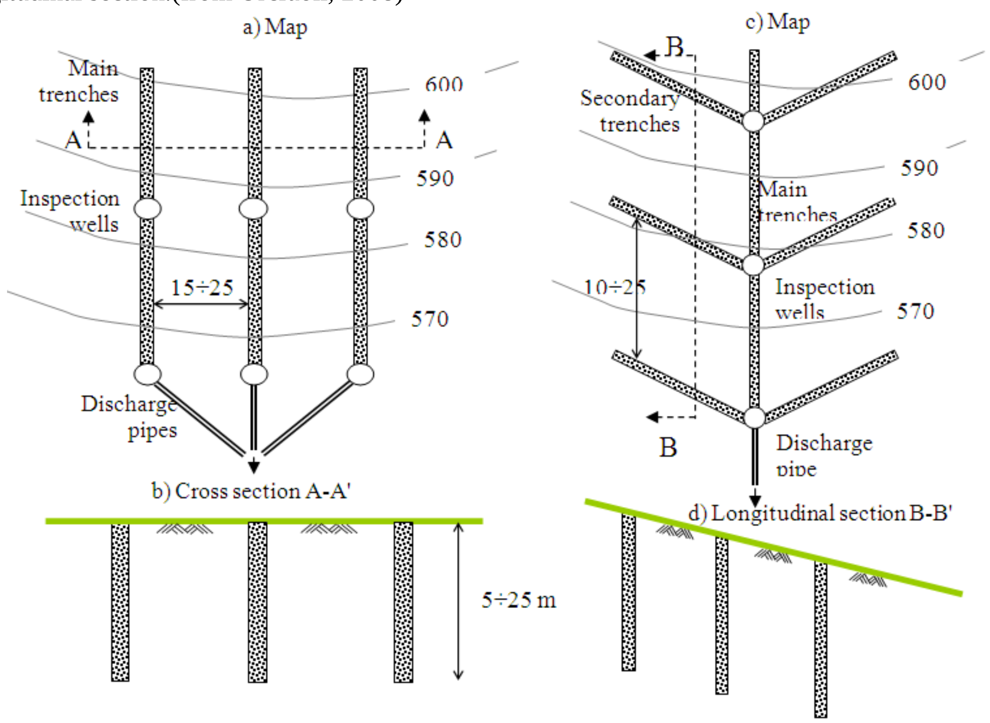


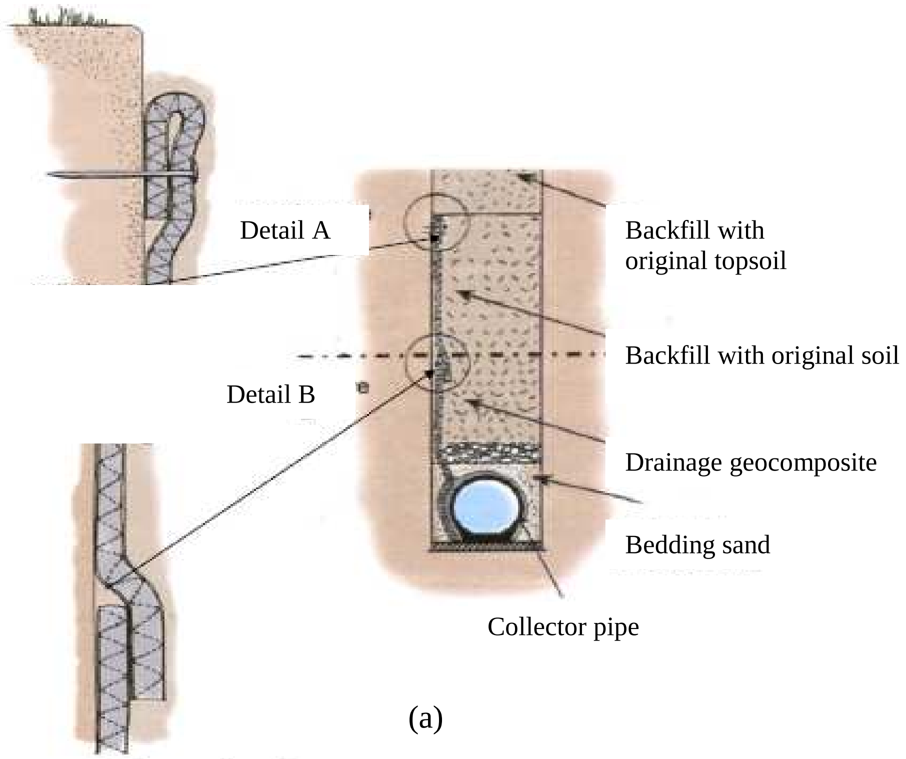
Figure 2: Grab shells used for trenches up to a depth higher than 5 m from the ground surface.

MITIGATION THROUGH REDUCTION OF HAZARD

4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE

4.2 DEEP TRENCHES FILLED WITH FREE-DRAINING MATERIAL

Fig. 3: a) Scheme of narrow trenches with geo-composit and pipes; b) Example of narrow trenches with geocomposite



(a)

(b)

Figure. 4: Construction of deep drainage trench by secant piles technique: a) first series of piles; b) odd-numbered piles

(a)

(b)

MITIGATION THROUGH REDUCTION OF HAZARD			
4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE			
4.2 DEEP TRENCHES FILLED WITH FREE-DRAINING MATERIAL			
APPLICABILITY			
Class	**Descriptor**	**Rating**	**Notes**
Type of movement (Cruden & Varnes, 1996)	Falls	0	Deep drainage is used to stabilize translational slides of large extension or rotational slides characterized by a deep slip surface.
	Topples	0	
	Slides	8	
	Spreads	2	
	Flows	6	
Material	Earth	8	Translational slides occur typically in fine-grained soils strongly altered and characterized by a permeability much higher than that of the underlying layer.
	Debris	6	
	Rock	4	
Depth of movement	Superficial (< 0.5 m)	8	The maximum depth for deep drainage system is 20-25m; therefore the best efficiency value is calculated at a depth less than that. As a consequence the depth of slip surface should not be more than 15-20m.
	Shallow (0.5 to 3 m)	8	
	Medium (3 to 8 m)	8	
	Deep (8 to 15 m)	4	
	Very deep (> 15 m)	0	
Rate of movement (Varnes, 1978)	Moderately to fast	6	The steady-state condition is attained some time after drainage construction (i.e. at the long term) in fact after drain installation, a transient phenomenon of equalization of pore pressures occurs. The drains are completely effective after such a delay and they represent the suitable mitigation method for very slow landslides.
	Slow	8	
	Very slow	8	
	Extremely slow	8	
Groundwater	Artesian	4	This system is suitable for lower shallow freatic water table.
	High	8	
	Low	4	
	Absent	0	
Surface water	Rain	6	The methods of analyzing the stabilization effect of drains commonly available in literature assume the presence of a film of water at ground surface. In areas where the weather is not very rainy, such as in Southern Europe, this assumption underestimates the effects of drains on slope stability. However the rain –water infiltration influences the system performance less than in the case in which shallow trenches are adopted.
	Snowmelt	6	
	Localized	0	
	Stream	0	
	Torrent	0	
	River	0	
Maturity		8	Technique and design process are well established and widely used in suitable conditions.
Reliability		7	The good working of drains depends strongly on the maintenance, possibly flushing of the perforated pipe. However the service is enough long.
Implementation		6	Some uncertainties about good construction of the system can exist because of the high depth to reach and the large spil volume involved.
Typical Cost		6	Deep drains are more costly than the surface drains because of the deep excavation and the large soil volume involved.

Note
Ratings are given on a scale of 1 to 10; the higher the grade, the most suitable is the specific method under consideration to use in landslides of the given characteristics, evaluated individually. Overall suitability to specific case under consideration may be obtained by a weighted average of these ratings, with user defined weights. Zero rating means "not applicable"

MITIGATION THROUGH REDUCTION OF HAZARD

4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE

4.2 DEEP TRENCHES FILLED WITH FREE-DRAINING MATERIAL

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MITIGATION THROUGH REDUCTION OF HAZARD

4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE

4.3 SUB-HORIZONTAL DRAINS (CONVENTIONAL DRILLING)

Description

Horizontal drains are used to stabilize deep landslides essentially characterized by circular slip surface. They are adopted in fine-grained soils and in fissured rocks. The schematic layout of horizontal drains, represented in typical longitudinal and cross sections, are shown in Figs. 1 a, b, c (Pun & Urciuoli, 2008).

Horizontal drains involve drilling holes in the ground, drilled with a tricone or drag bit (Figs. 2, 3). The diameter of the hole is usually 100-120 mm. During drilling, flushing fluid such as bentonite mud, polymers, foam, water or air is required to reduce friction and aid the removal of the cuttings. A PVC slotted pipe, protected by a geo-textile to avoid the clogging with fine materials, is inserted in the hole (Fig. 4). The maximum length of horizontal pipes is around 100 m, but in some cases it has been possible to reach 300 m. Deposits of calcium, salts and iron oxide can block horizontal drains during operation; therefore regular maintenance by flushing the pipes with a high pressure water jet, should be programmed. In absence of maintenance, drain pipes cannot keep functioning for a long time (maximum 15-20 years). A good practice to reduce precipitation of calcite inside pipes consists of drilling the hole at an inclination slightly above horizontal, such that the pipe is not continuously submerged. Conversely, there are other chemical phenomena, favored by bacterial activity, that are due to aeration (Walker & Mohen 1987). At the portion of the horizontal drain near to the slope surface, it is recommended to use a 3-6 m long un-perforated pipe, grouted all around with cement, to prevent the penetration of tree roots into the pipe, which could block the water flow (Fig. 4). The timerequired for installation is approximately 100-200 m per day.

Design

The design of horizontal drains can be carried out by using numerical analyses or easily by adopting design charts available in literature (see Di Maio et al.1988, Desideri et al. 1997, Pun & Urciuoli 2008).

In the first case drainage work is analysed by means of numerical codes (DEM or FEM) and the problem may be solved by taking soil stratigraphy and heterogeneity into account and by assuming a water flux depending on the climate condition at the upper boundary. In this way, pore pressures can be calculated along the critical sliding surface; then they can be used in slope stability analysis.

In the second case, non-dimensional charts obtained for homogeneous soil and very simple geometric schemes are used to estimate pore pressure, lowered by drains. Design charts are a general tool useful for general conditions: they cannot consider hydraulic conditions at ground surface according to a seasonal trend, which necessarily depends on typical climatic features of the region. In fact the methods for analyzing the stabilization effect of drain trenches commonly available in the literature assume the presence of a film of water at the ground surface. The major use of design charts consists of obtaining the geometric configuration from the global efficiency of the drainage system that is determined as a function of the pore water distribution that guarantees the safety factor chosen by the designer.

Based on numerical analyses, Desideri et al.1998 proposed design charts for drains installed from the ground-surface, with the drain rows placed at the distance S along the maximum slope direction (if they are installed on two or more alignments); the distance along the direction normal to slope is indicated by i. L is the length of the slope where pore pressures are reduced by draining, D is the depth of the plane parallel to the ground surface on which the efficiency is evaluated, X_{pd} is the relative position of the drainage system respect the lower end of the longitudinal section L (Fig 5 a,b).

In the analysis, the following hypothesis are assumed:

- infinite slope;
- homogeneous soils and isotropic permeability;
- presence of a film of water at the ground surface;
- two-dimensional flow conditions (by assuming values of the ratio i/l<2);
- constant ratio d/l=0.02;
- flow parallel to the ground surface, as initial conditon.

Design charts have been developed by the authors for different slope angles, as a function of the ratios l/L, X_{pd}/L, S/L,. for one and two rows of drains. The design charts allow to obtain:

- the optimum design of system;
- the system efficiency;
- the time at which 90% and 50% of efficiency is reached.

Figure 1:: Horizontal drains inserted from the ground surface: a) Plan, b) Longitudinal section, c) Cross section. (from Pun and Urciuoli, 2008).

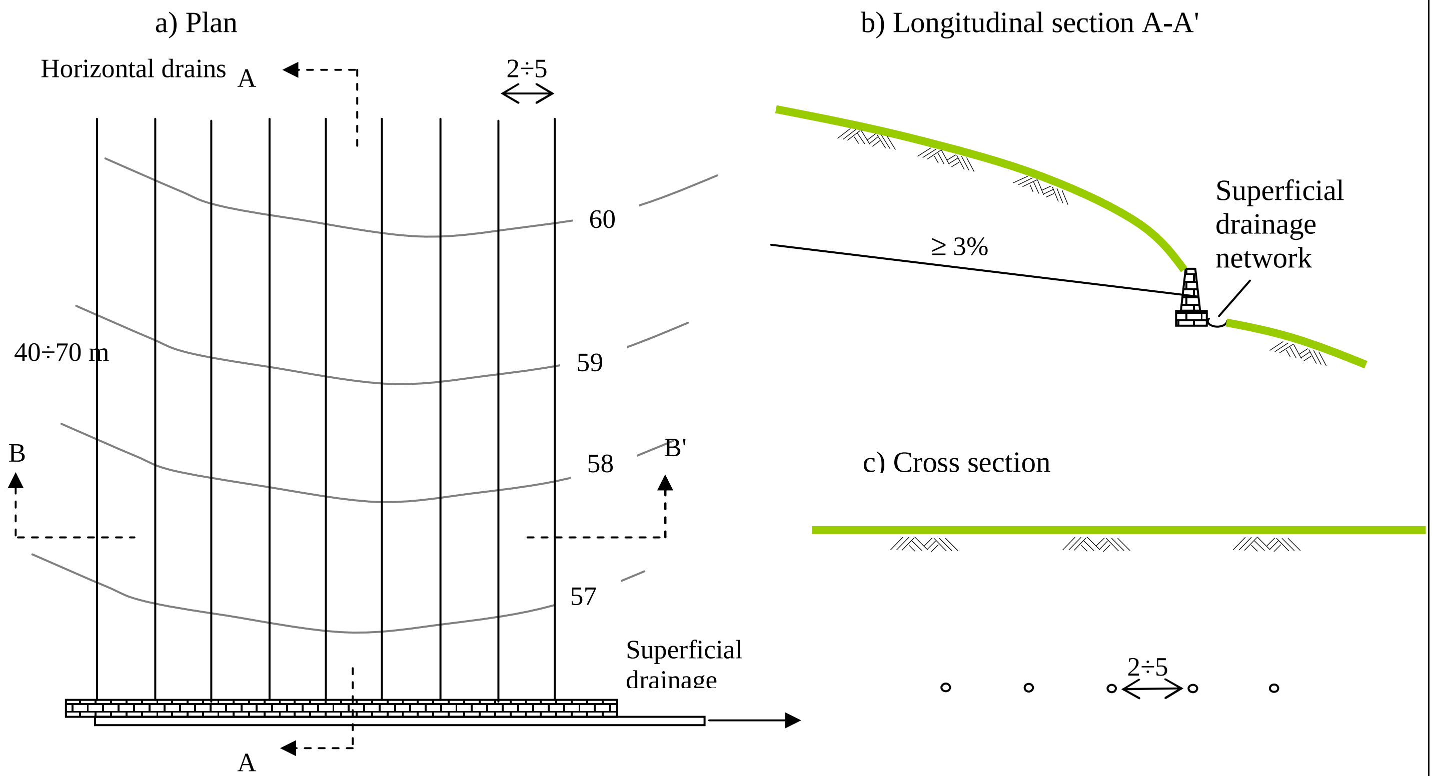
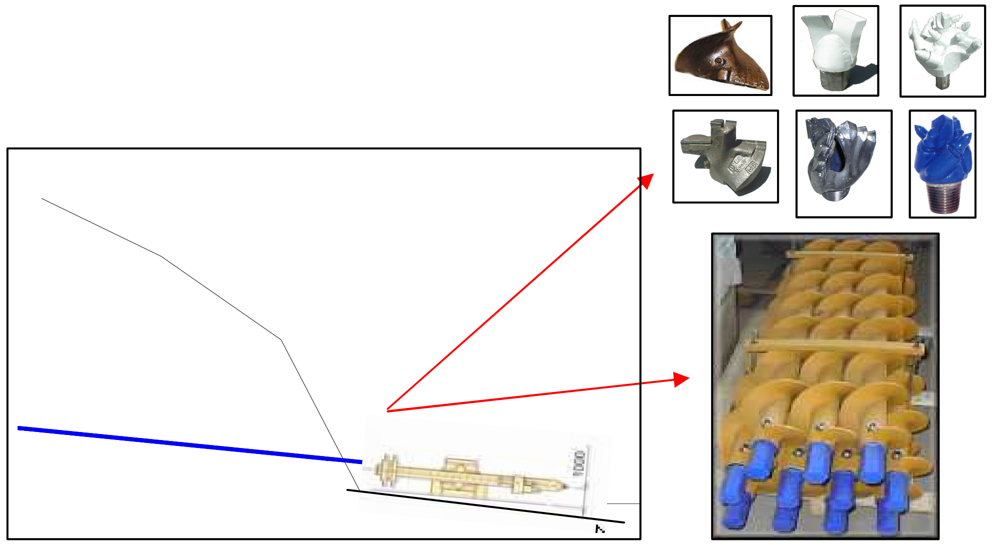


Figure 2: Rotary drilling for horizontal drains. Schematic section and drilling tools



MITIGATION THROUGH REDUCTION OF HAZARD

4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE

4.3 SUB-HORIZONTAL DRAINS (CONVENTIONAL DRILLING)

The first step of the design procedure is to determine the maximum factor of safety corresponding to the atmospheric pore pressure distribution in the subsoil and then the increment of the factor of safety (relative to its initial value) that is necessary to assure a suitable level of safe of the slope. These two values of safety factors are the ingredients for the calculation of the long-term efficiency of the drain system. The following step is to design the geometric configuration of drains that will result in the required pore pressure change. A single level of drains is assumed initially. The values of L, D, X_{pd} are assigned and by using Figures 6c, d, 7c, d the lenght of drains, l, and then the efficency at long term are obtained. In this way it is possible to evaluate if the efficiency of the hypothized drain system is larger than the required value. If necessary, an increase in system efficiency can be achieved by increasing the lenght of the drains, but no significant benefits are obtained for values higher than l = 4 -5 D. The values of the time factor corresponding to 50% and 90% of efficiency are obtained from Figures 6a, b, 7a, b, .
If the results do not satisfy the design problem (too long to achieve efficiency, low safety factor, etc..) a system of drains installed on two levels can be considered. A value of E_∞ is fixed and l, X_{pd}, and S are determined by using Figures 8-9.

Figure 5: Drain installation scheme: a) isometric view; b) longitudinal section (source: Desideri et al.,1998)

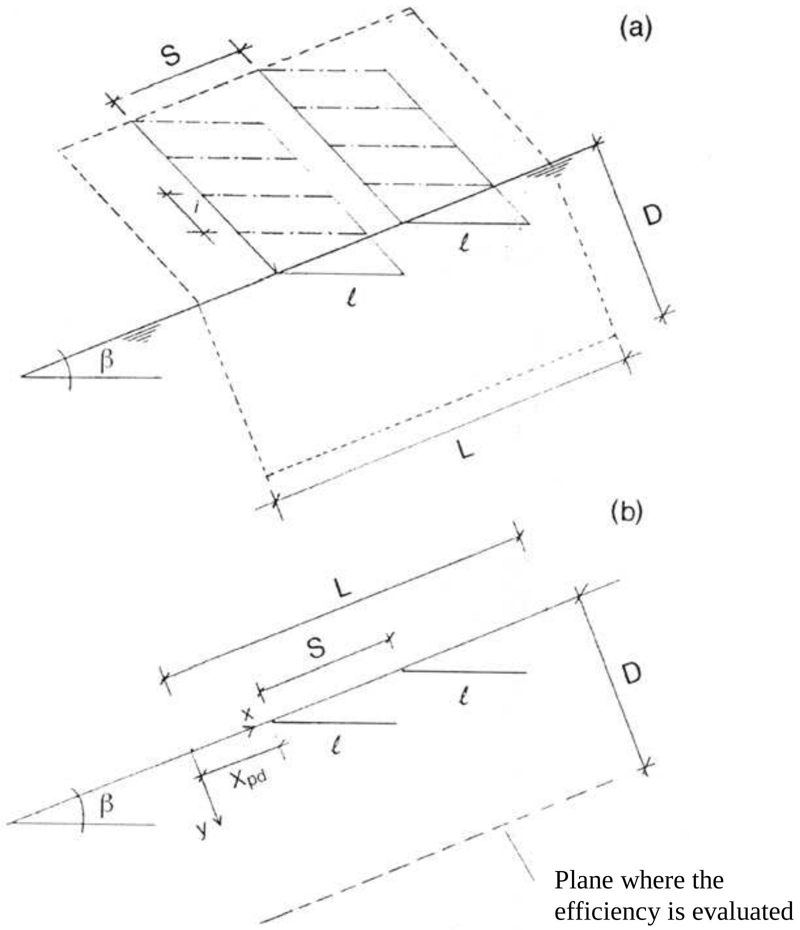
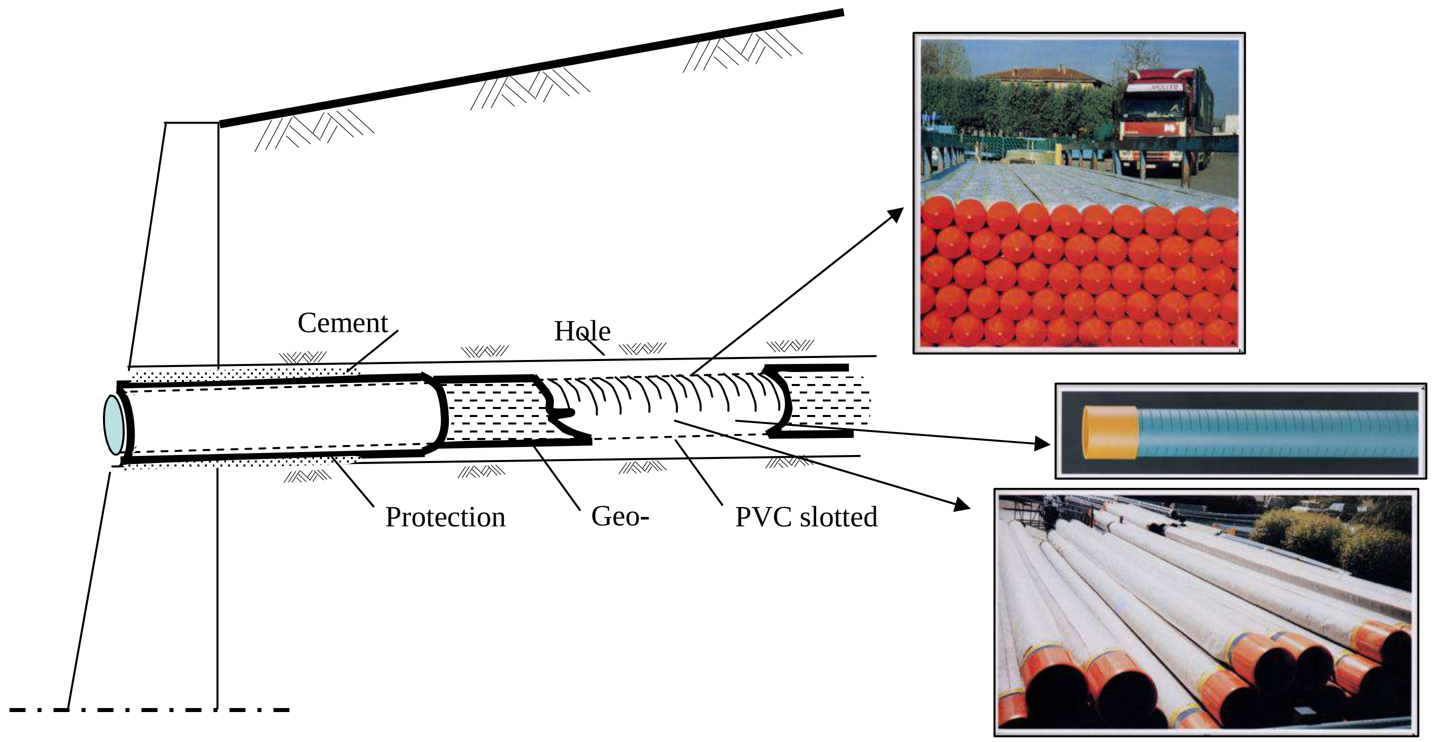


Figure 3:: Drilling of boreholes

Figure 4: Schematic arrangement of a horizontal drain



MITIGATION THROUGH REDUCTION OF HAZARD

4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE

4.3 SUB-HORIZONTAL DRAINS (CONVENTIONAL DRILLING)

Figure 6: Design charts for slope angle = 30°: a) time factor for E=50%, b) time factor for E=90%,
c) E∞ (at long term) as a function of lengnt l, d) X_{pd} as a function of lenght l (source: Desideri et al.,1998)

α=30°

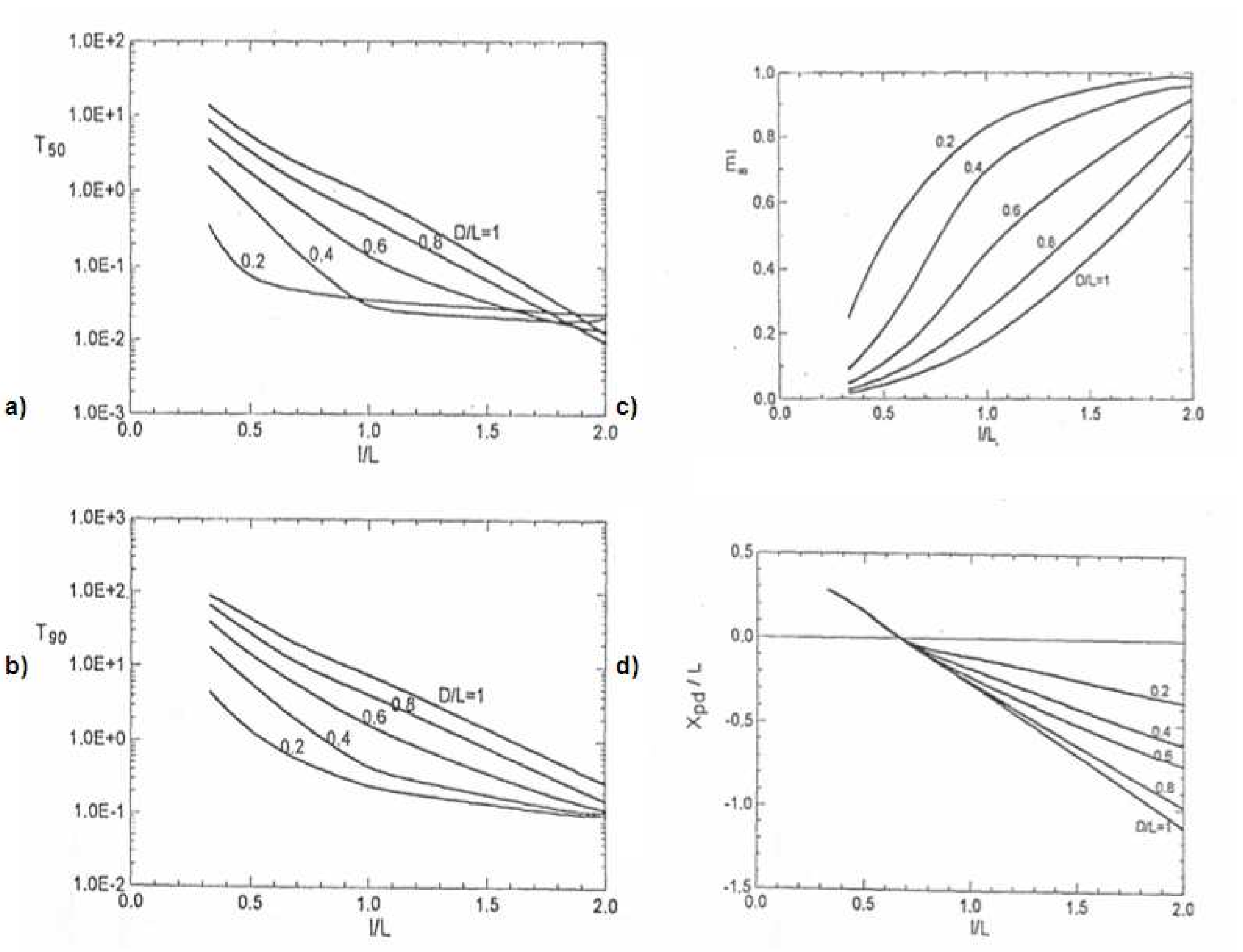
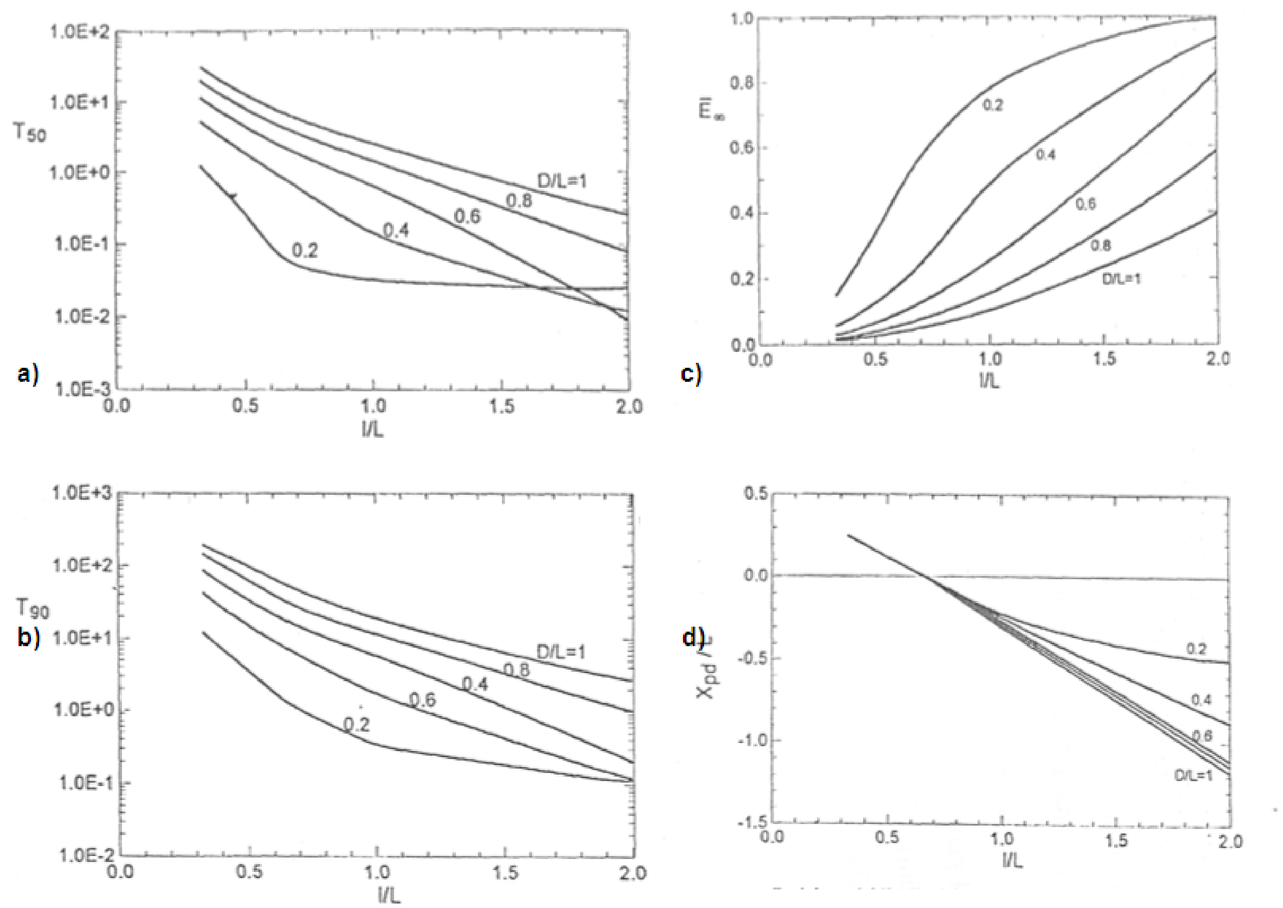


Figure7: Design charts for slope angle = 20°: a) time factor for E=50%, b) time factor for E=90%,
c) E∞ (at long term) as a function of lengnt l, d) X_{pd} as a function of lenght l (source: Desideri et al.,1998)

α=20°



MITIGATION THROUGH REDUCTION OF HAZARD

4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE

4.3 SUB-HORIZONTAL DRAINS (CONVENTIONAL DRILLING)

Figure 8: Design charts for slope angle = 20° and two rows of drains: a) S as a function of l, b) E∞ (at long term) as a function of l , c) time factor for E=50%, d) X_{pd} as a function of lenght, l , e) time factor for E=90%. (Desideri et al.,1998)

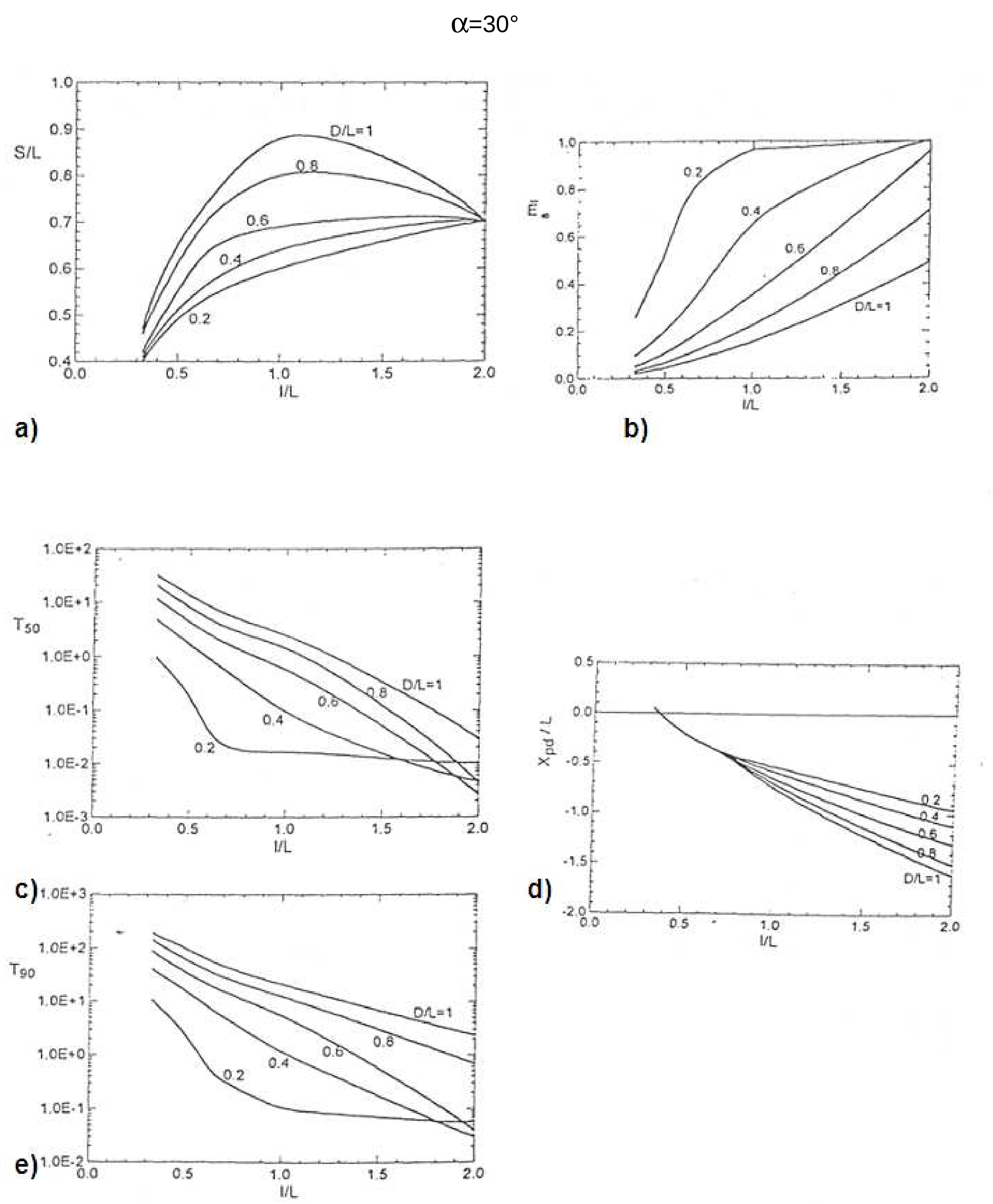
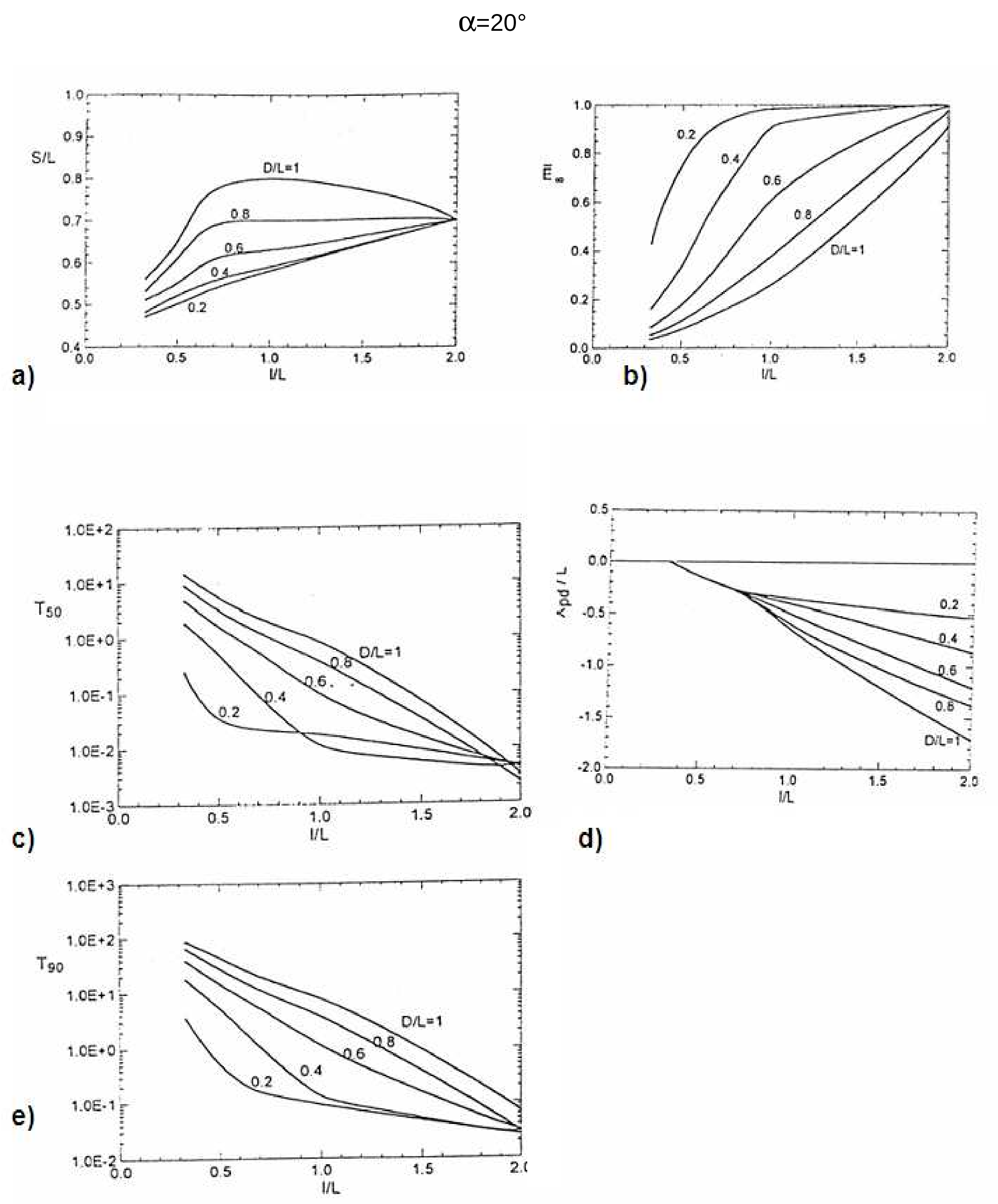


Figure 9: Design charts for slope angle =30° and two rows of drains: a) S as a function of l, b) E∞ (at long term) as a function of l , c) time factor for E=50%, d) X_{pd} as a function of lenght, l , e) time factor for E=90%.(source: Desideri et al.,1998)



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4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE

4.3 SUB-HORIZONTAL DRAINS (CONVENTIONAL DRILLING)

Figure 10: Phases of construction of sub-horizontal drains

MITIGATION THROUGH REDUCTION OF HAZARD

4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE

4.3 SUB-HORIZONTAL DRAINS (CONVENTIONAL DRILLING)

APPLICABILITY

Class	Descriptor	Rating	Notes
Type of movement (Cruden & Varnes, 1996)	Falls	2	Horizontal drains are used to stabilize deep slides essentially characterized by circular slip surface and with a high slope angle of the ground surface.
	Topples	2	
	Slides	6	
	Spreads	2	
	Flows	4	
Material	Earth	4	They are adopted in fine-grained soils and in fissured rocks as well.
	Debris	8	
	Rock	4	
Depth of movement	Superficial (< 0.5 m)	0	The horizontal drain system is suitable for deep slip surfaces.
	Shallow (0.5 to 3 m)	2	
	Medium (3 to 8 m)	6	
	Deep (8 to 15 m)	6	
	Very deep (> 15 m)	4	
Rate of movement (Varnes, 1978)	Moderately to fast	2	The steady-state condition is attained some time after drainage construction (i.e. at the long term) in fact after drain installation, a transient phenomenon of equalization of pore pressures occurs. Drains are completely effective after such a delay and they represent the suitable mitigation measure for very slow landslides.
	Slow	6	
	Very slow	8	
	Extremely slow	8	
Groundwater	Artesian	4	This system is suitable for deep freatic water table.
	High	6	
	Low	8	
	Absent	0	
Surface water	Rain	4	Horizontal drains are not suitable to drain shallow water.
	Snowmelt	4	
	Localized	0	
	Stream	0	
	Torrent	0	
	River	0	
Maturity		7	Technique and design process are well established and widely used in suitable conditions.
Reliability		6	Necessary to flush the pipes with high pressure water jets for good operation. The most frequent problems are: fouling, deterioration of the final collector, changing of the water path.
Implementation		6	Difficult to have good installation of the pipe, especially if very long; it is good practice to drill the hole slightly inclined to allow gravity drainage.
Typical Cost		7	Lless costly than other types of stabilization works and suitable for a large number of cases, especially when the landslide is very deep and structural measures are not effective..

Note
Ratings are given on a scale of 1 to 10; the higher the grade, the most suitable is the specific method under consideration to use in landslides of the given characteristics, evaluated individually. Overall suitability to specific case under consideration may be obtained by a weighted average of these ratings, with user defined weights. Zero rating means "not applicable"

MITIGATION THROUGH REDUCTION OF HAZARD

4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE

4.3 SUB-HORIZONTAL DRAINS (CONVENTIONAL DRILLING)

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MITIGATION THROUGH REDUCTION OF HAZARD

4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE

4.4 SUB-HORIZONTAL DRAINS (DIRECTIONAL DRILLING)

Description

Horizontal Directional Drilling (HHD), is an innovative technique adapted from the drilling technology used usually in the petrochemical industry, for the installation of underground utilities where conventional open-trenching solutions are inappropriate or not permitted, such as under rivers, railways, highways, in protected areas (national parks, urban areas of historical importance) or in densely populated residential areas (Figure 1).

This technology is currently also used for slope stabilization, to lay the drain pipes instead of the conventional drilling; it can be used in geological conditions ranging from soft to very hard formations.

The **process** starts with the construction of the receiving hole and the entrance pits. These pits will allow the drilling fluid to be collected and reclaimed to minimize cost and to prevent excessive waste. The first stage drills a pilot hole on the designed path (Fig. 2a) and the second stage enlarges the hole by passing a larger cutting tool known as the back reamer (Figs. 2b, 3b). The reamer's diameter depends on the size of the pipe. Throughout the drilling and reaming process the drilling is done with the help of a viscous drilling fluid. It is a mixture of water and, usually, polymer continuously pumped to the cutting head or drill bit to facilitate the removal of cuttings, stabilize the bore hole, cool the cutting head, and lubricate the passage of the product pipe. The third stage places the drain in the enlarged hole by means of the drill steel and is pulled behind the reamer (Fig. 3c) to allow centering of the pipe in the newly reamed path (Fig. 2c).

The **equipment** used in a directional drill operation (Figs. 3, 4, 5, and 6) depends on the size of the pipe, length of the run, and surrounding locations. For the large bores, a 100,000 pound pulling power drill is used with a reclaimer, excavator, and multiple pumps and hoses to move the fluid. The drilling steel is a 3-in. diameter pipe with male and female threads (Fig. 4). The head of the operation comes in multiple designs and depends on the rock or soil being penetrated. The drilling head (Fig. 6) has multiple water ports to allow removal of material. A talon bit involves the diamond tipped cutters. These allow for steering and cutting the material. Another head type is a mud-motor which is used in rocky landscapes (Fig. 6).

Typically a small two-person crew is required including a drill operator and a tracker. The tracker directs the progress of the drill by using a hand held device that gathers data from a sonde located in the drill head just behind the bit. The advantages of this system are:

- the size of the worksite consists of two small entry and exit pits;
- the drain may be laid at the desired depth with no risk to the operator;
- the bore path can be directed to avoid buried obstacles or other utilities, or to follow an angled trajectory according to the particular requirements of the bore design;
- the installation is faster and safer with no need to back-fill the excavation.

An experimental application at Barton-on-Sea, UK, proved very successful. The drains were drilled from a starter pit in very stiff clay some distance away from the toe of the unstable seacliff. Once drilling had penetrated sufficiently below the toe of the cliff, the directional drilling was made to turn upwards to come out onto the main plateau at the top of the cliff, where the reamer and the perforated pipe werea fixed to the drillstring and pulled back to the starter pit. This arrangement allowed the drains to intercept several perched water tables in the stratified soil profile and to discharge by gravity. The minimal intrusiveness of the technquie is an added bonus, allowing installation in envioronmentally sensiive locations with minimal disruption.

Design

The design of horizontal drains can be carried out by using numerical analyses or easily by adopting design charts available in literature (Di Maio et al.1988, Desideri et al. 1997, Pun & Urciuoli 2008). See the section 4.3.

However when this technology is used apart from the design of drains (length, diameter, number, and interspace), it is important that the work area at entry and exit is adequate and safe and to plan bore path with adequate separation from utilities and obstacles.

Figure. 1: Different applications of HHD technology. Slope stabilization highlighted in red

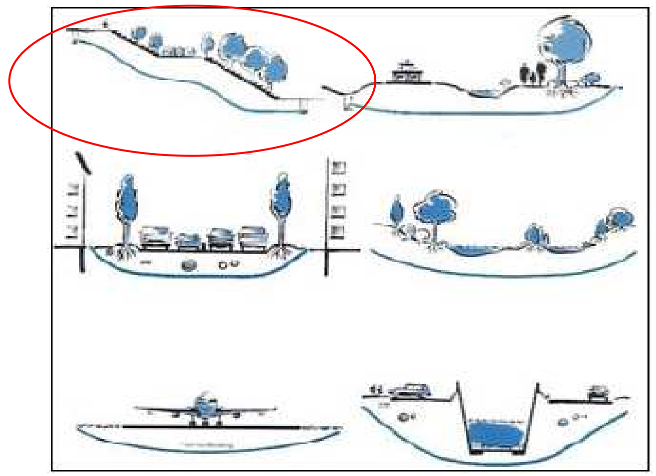
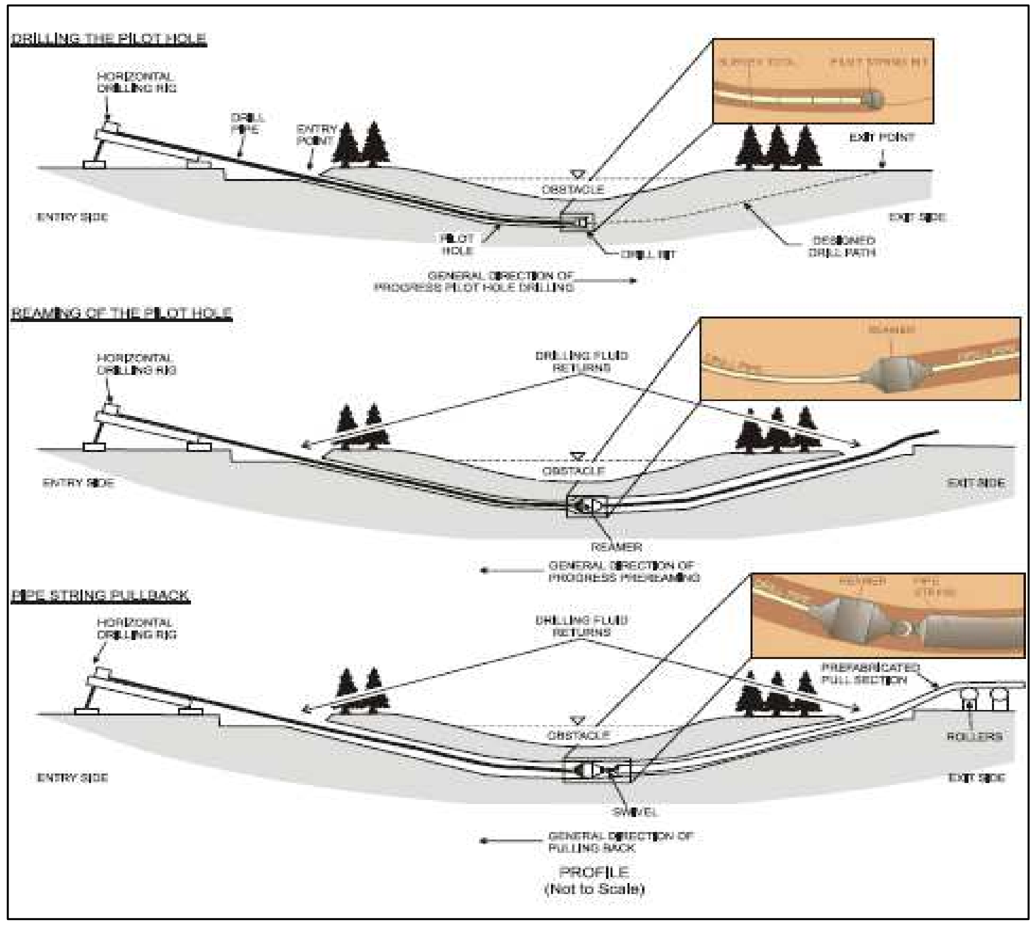


Figure 2: The phases of HDD construction: a) drilling the pilot hole, b) reaming of the pilot hole, c) pipe string pull - back.

a)

b)

c)



MITIGATION THROUGH REDUCTION OF HAZARD

4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE

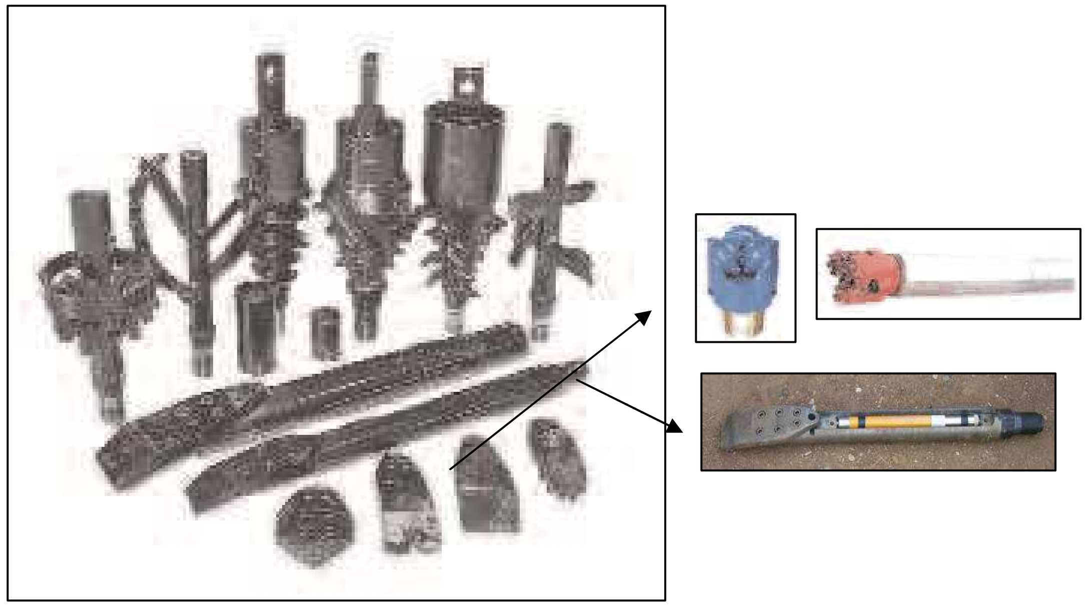
4.4 SUB-HORIZONTAL DRAINS (DIRECTIONAL DRILLING)

Figure 3: Power drilling machine

Figure 4: Rods

Figure 5: Pipe lines

Figure. 6: Different applications of HHD technology.



MITIGATION THROUGH REDUCTION OF HAZARD

4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE

4.4 SUB-HORIZONTAL DRAINS (DIRECTIONAL DRILLING)

Figure 7: Laying of drain by means of directional drilling technology (HHD) at Montaldo di Cosola (AL), Italy (source: Capaccetta, 2006)

MITIGATION THROUGH REDUCTION OF HAZARD

4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE

4.4 SUB-HORIZONTAL DRAINS (DIRECTIONAL DRILLING)

APPLICABILITY

Class	Descriptor	Rating	Notes
Type of movement (Cruden & Varnes, 1996)	Falls	2	Horizontal drains are used to stabilize deep landslides essentially characterized by circular slip surface.
	Topples	2	
	Slides	6	
	Spreads	4	
	Flows	4	
Material	Earth	4	The Horizontal Directional Drilling technology can be applied to several soil types such as clay, sandy and limey soil, and rocky ground.
	Debris	8	
	Rock	4	
Depth of movement	Superficial (< 0.5 m)	0	These drainage system can reach very deep slip surface through any type of path.
	Shallow (0.5 to 3 m)	0	
	Medium (3 to 8 m)	6	
	Deep (8 to 15 m)	8	
	Very deep (> 15 m)	8	
Rate of movement (Varnes, 1978)	Moderately to fast	2	The steady-state condition is attained some time after drainage construction (i.e. at the long term) in fact after drain installation, a transient phenomenon of equalization of pore pressures occurs. Drains are completely effective after such a delay and they represent the suitable mitigation measure for very slow landslides.
	Slow	6	
	Very slow	8	
	Extremely slow	8	
Groundwater	Artesian	4	This system is suitable for deep freatic water table.
	High	6	
	Low	8	
	Absent	0	
Surface water	Rain	4	Horizontal drains are not suitable to drain shallow water.
	Snowmelt	4	
	Localized	0	
	Stream	0	
	Torrent	0	
	River	0	
Maturity		6	Technique and design process are sufficiently established.
Reliability		6	It's necessary flushing the pipes with a high pressure water jet for a good working.
Implementation		7	Drain alignement can be adapted to avoid obstacles and buildings. Easily implemented with 2 man crew. Faster and safer than other methods, with no need to enter or backfill threnches, but requires specalist equipment.
Typical Cost		6	The cost of this type of drilling is 5-7 times higher than the conventional drilling.

Note
Ratings are given on a scale of 1 to 10; the higher the grade, the most suitable is the specific method under consideration to use in landslides of the given characteristics, evaluated individually. Overall suitability to specific case under consideration may be obtained by a weighted average of these ratings, with user defined weights. Zero rating means "not applicable"

MITIGATION THROUGH REDUCTION OF HAZARD

4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE

4.4 SUB-HORIZONTAL DRAINS (DIRECTIONAL DRILLING)

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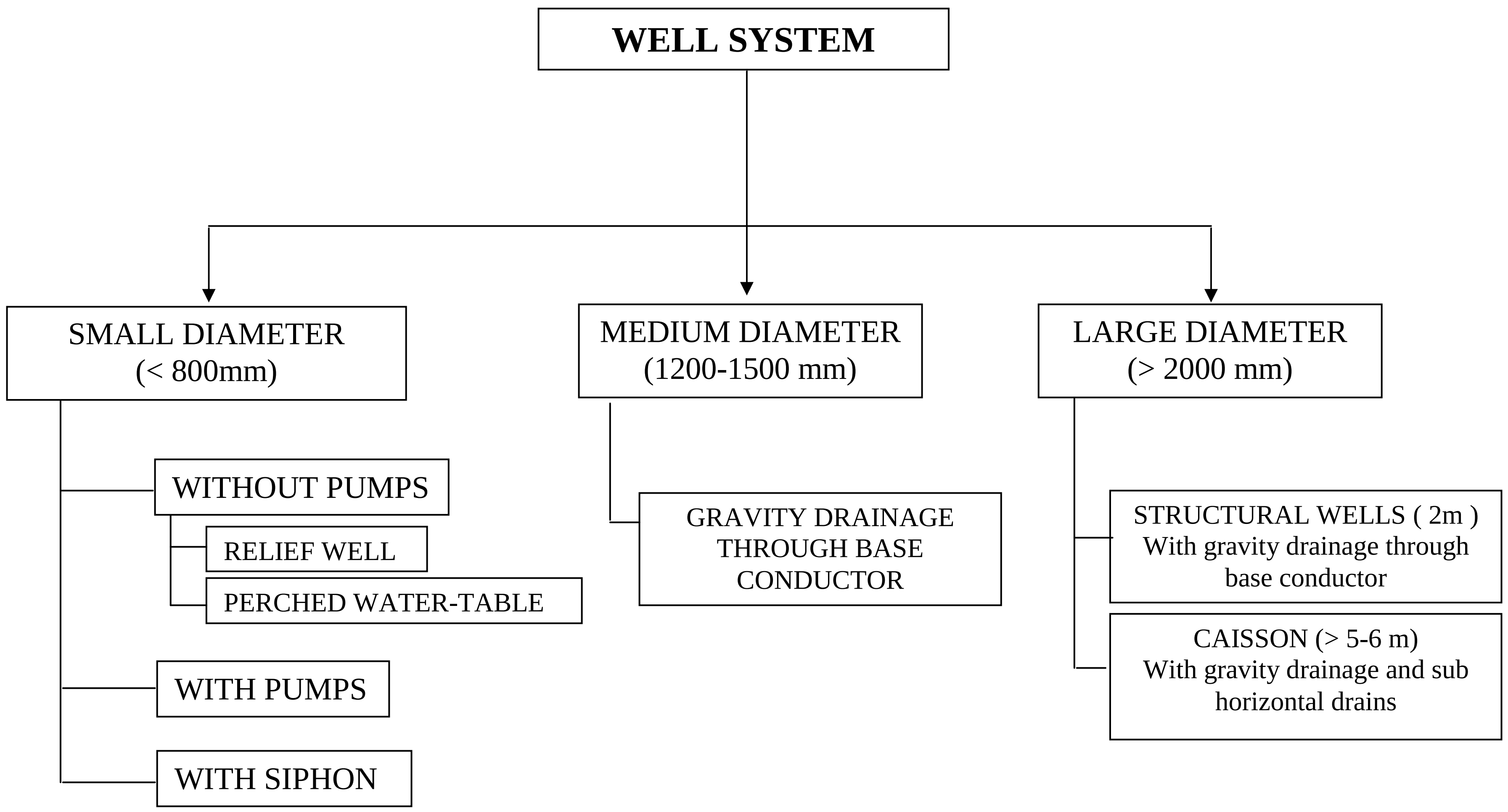
4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE

4.5 CLASSIFICATION OF WELLS



MITIGATION THROUGH REDUCTION OF HAZARD

4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE

4.5.1 SMALL AND MEDIUM DIAMETER WELLS - GENERAL ASPECTS

Description

Wells are used in deep landslides. They are necessary when the water table or landslide surface are deep and when the soil is not homogeneous but is characterized by horizontal layers of different permeability, among which more permeable ones must be captured. Wells are usually divided in (Fig. 1).

- wells of small diameter (< 800 mm);
- wells of medium diameter (1200 - 1500 mm);
- wells of large diameters (> 2m) or structural wells.

Small diameter wells can work without pumps or by means of pumps or siphons. The medium and large diameter wells usually allow the drawdown of the water by means of gravity drainage through the bottom and of the sub horizontal drains (well diameter >3 m) (Fig.1). The cost is higher than the other drainage systems, especially when pumping is necessary. The construction of small and medium diameter wells is the same and is described below. Large diameter wells are described in fact sheets 4.5.6 and 4.5.7.

Technology for small and medium diameter wells

Small and medium diameter wells consist of a drilled hole; a screen or slotted pipe section to allow entrance of ground water; a bottom plate; a filter to prevent entrance and ultimate loss of aquifer material; a riser to conduct the water to the ground surface; a check valve to allow escape of water and prevent back flooding and entrance of foreign material; backfill to prevent recharge of the formation by surface water; and a cover and some type of barricade protection to prevent vandalism and damage to the top of the well by maintenance crews, livestock, etc (Fig. 2).

The hole should be vertical so that the screen and riser can be installed straight and plumb. The hole is drilled large enough to provide a minimum thickness of 10 – 15 cm, depending on the gradation, of the filter material. The methods of providing an open boring in the ground are:

- Standard Rotary Method (Fig. 3b)**:** Standard rotary drilling consists of rotating a cutter bit against the bottom of a boring, while a fluid is pumped down through the drill pipe to cool and lubricate the bit and return the cuttings up the open hole to the ground surface. The fluid must be biodegradable, organic; no bentonitic clays are used in the drilling fluid.
- Reverse-Rotary Method: This method is generally considered to provide the most acceptable drill hole and should be used whenever possible for the installation of permanent wells. In the reverse-rotary method, the hole for the well is made by rotary drilling, using a similar cutting process as employed in standard rotary drilling except the drilling fluid is pulled up through the drill pipe by vacuum and the drilling fluid reenters the top of the open boring by gravity. Soil from the drilling is removed from the hole by the flow of drilling fluid circulating from the ground surface down the hole and back up the hollow drill stem from the bit.
- Bailing and Casing (Fig. 3a): Where standard or reverse-rotary drilling is not successful, especially in caving alluvial sands and unconsolidated palaeochannel deposits, an equally acceptable method of drilling consists of bailing while driving a steel casing into the hole to stabilise the boring walls. This method is economical in some materials, and it does not inject deleterious materials into the formation. Loose to medium dense, clean, granular materials can be bailed economically. Thin layers of cohesive materials, or cemented materials within the formation, can preclude the advance by bailing and may also produce smear along the sides of the drill hole which could impair free flow into the well.
- Bucket Augers: Under certain conditions drill holes for relief wells can be made with a bucket auger. The method has been successfully employed where cobbles up to 254 mm have been encountered. A bucket with side cutters is employed, and only water is used as the drilling fluid.

Once the boring is completed and the tools withdrawn, the well screen and riser pipe can be constructed at the site in **varying** lengths. The lengths of screen are connected together as they are lowered into the hole. The riser and screen sections should be centred in the drill hole by means of appropriate centring devices to facilitate a continuous filter around the well screen. Then the filter may be placed. A tremie should be used to maintain a continuous flow of material and thus minimise segregation during placement. After the tremie pipe or pipes have been lowered to the bottom of the hole, they should be filled with filter material and then slowly raised to keep them full of filter material at all times.

Figure. 1) Classification of wells

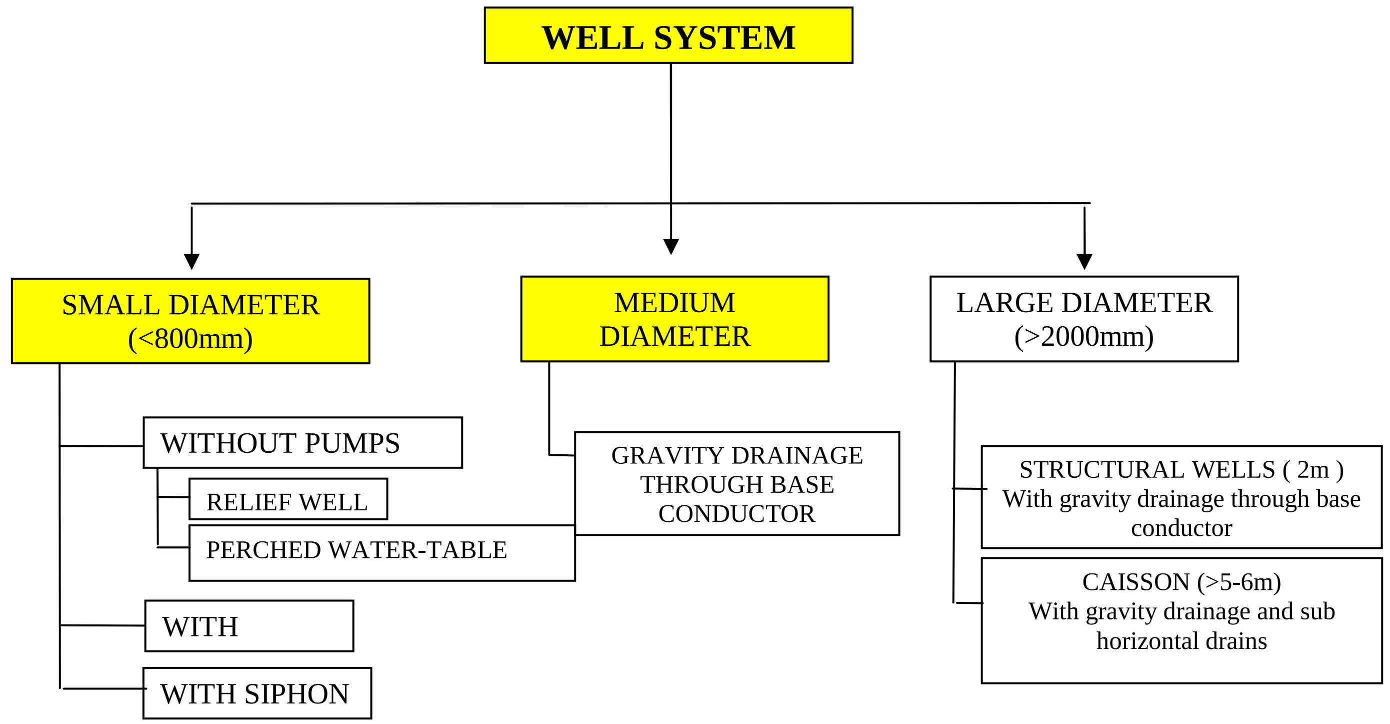
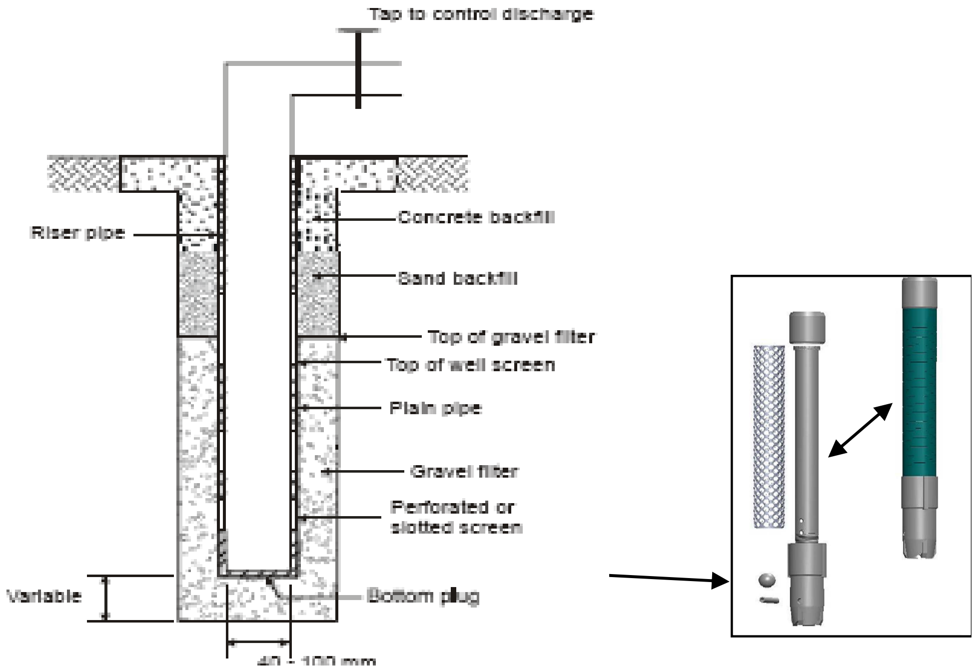


Figure 2) Typical small diameter well and well screen



MITIGATION THROUGH REDUCTION OF HAZARD

4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE

4.5.1 SMALL AND MEDIUM DIAMETER WELLS - GENERAL ASPECTS

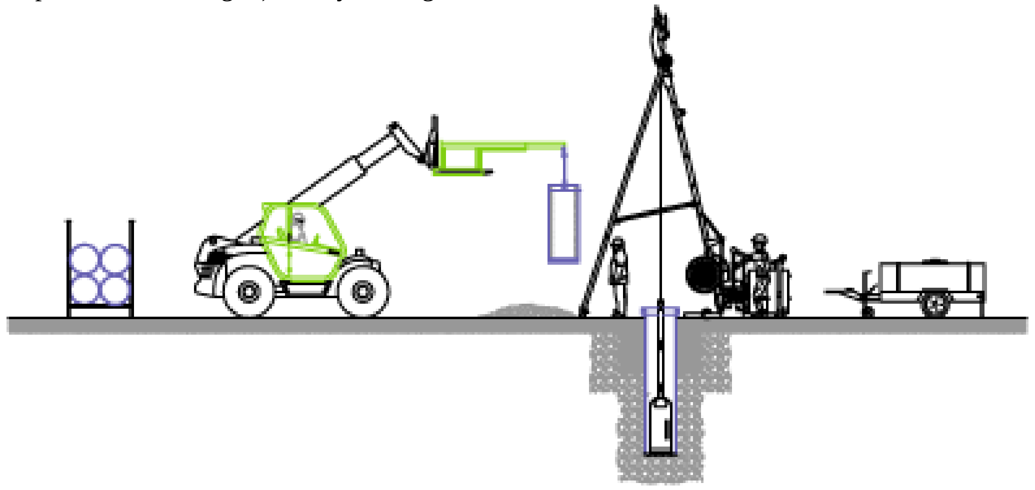
Extending the filter material at least 60 cm above the top of the screen will depend on the depth of the well to compensate for settlement during well development. The level of drilling fluid or water in a reverse-rotary drilled hole must be maintained at least 2 m above the natural ground-water level until all the filter material is placed. If a casing is used, it should be pulled as the filter material is placed, and the bottom of the casing kept 60 - 300 mm below the top of the filter material. Development procedures include both chemical and mechanical processes. Development of a well should be accomplished as soon after the hole has been drilled as practicable. Delay in doing this procedure may prevent a well being developed to the efficiency assumed in design.

Chemical development is applied usually in the case where special drilling fluids are utilised and chemicals are injected into the well to aid in the dissolution of the residual drilling fluid in the filter. After the chemicals have been dispersed, the well should be pumped and the effluent checked to ensure that the drilling fluid has completely broken down. The purpose of mechanical development is to remove any film of silt from the walls of the drilled hole and to develop the filter immediately adjacent to the screen to permit an easy flow of water into the well. The result of proper development is the grading of the filter from coarsest to finest extending from the well. The effect of proper development is an increase in the effective size of the well, a reduction of entrance losses into the well, and an increase in the efficiency of the well. Basically there are three methods used in development: a) Water Jetting, b) Surging, c) Pumping.
During the development process, sand and silt will be brought into the well. When the depth of sand collected in the bottom of the screen reaches 30 cm, it should be removed by bailing. The remainder of the hole should be filled with either a cement-bentonite mixture tremied into place or concrete. In both cases, a 30 cm layer of concrete sand or excess filter material should be placed on top of the filter before placement of grout or concrete. A tremie equipped with a side deflector will prevent jetting of a hole through the sand and into the filter.

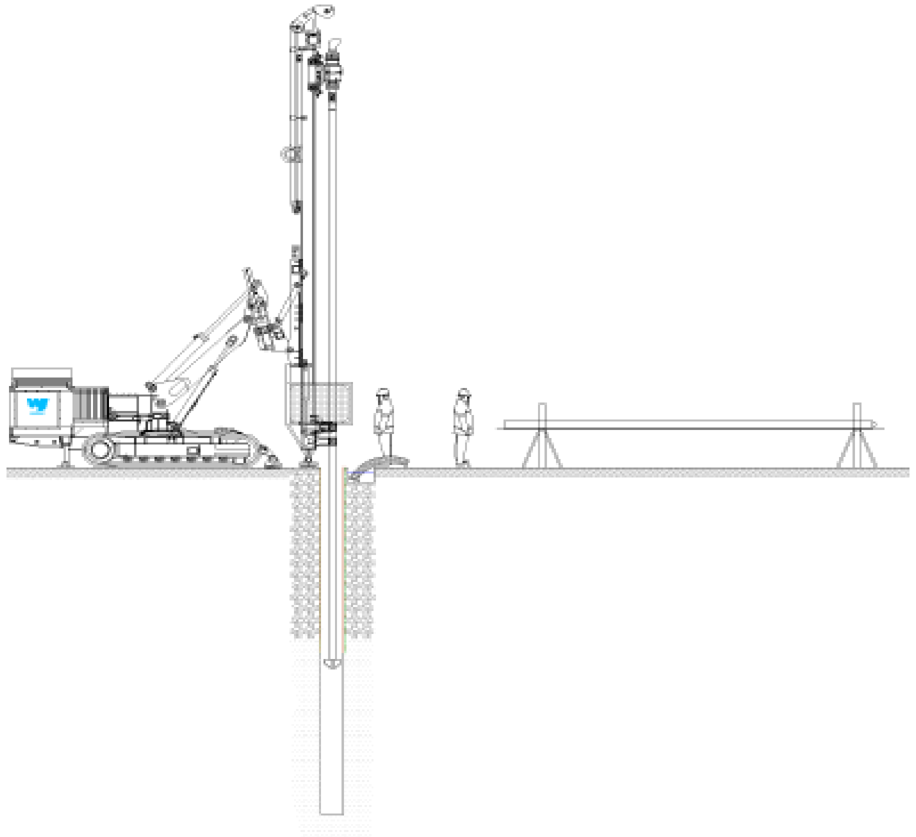
Materials for wells
Well screen (fig.2): Commercially available well screens and riser pipes are fabricated from a variety of materials such as black iron, galvanised iron, stainless steel, brass, bronze, fibreglass, polyvinyl chloride (PVC), and other materials. How well a material performs with time depends upon its strength, resistance to damage by servicing operations, and resistance to attack by the chemical constituents of the ground water. PVC appears to be completely stable, and it is easy to handle and install; however it is a relatively weak material and easily damaged. A variety of slot types are available in most types of well screens. PVC screens with open slots of varying dimensions consisting of a series of saw cuts are typically available. The size of the individual openings in a well screen is dictated by the grain size of the filter. The openings should be as wide as possible, yet sufficiently small to minimise entrance of filter materials. The open area of a well screen should be sufficiently large to maintain a low entrance velocity of less than 3 cm per second at the design flow. In general, the slot width (or hole diameter) of the screen should be equal to or less than the 50% size of the finest gradation of filter.
Filter: The filter gradation must meet the stability requirement that the 15% size of the filter should be not greater than five times the 85% size of the aquifer materials. The design should be based on the finest gradation of the foundation materials, excluding zones of unusually fine materials where blank screen sections should be provided. If the aquifer consists of strata with different grain size bands, different filter gradations should be designed for each band. Each filter gradation must also meet the permeability criterion that the 15% size of the filter should be more than three to five times the 15% size of aquifer sands. Either well graded or uniform filter materials may be used. The filter should consist of natural material made up of hard durable particles.

Well-characteristic curve
Pumping tests are necessary to obtain: (a) well-characteristic curve and (b) hydrogeologic characteristics of aquifer (permeability, K , trasmissivity, T, etc...). The well-characteristic curve is the relation between the decreasing water level in the well respect the initial piezometric level at equilibrium and the flow pumping, and in particular to know the optimal flow to pump. In order to stabilize a slope, if the decreasing of the piezometric level is realized by means of wells, the characteristic curve provides the flow to remove from aquifer to reach that ground-water level. The well-characteristic curves are shown in figures 4a and 4b, respectively for freatic and artesian aquifer.

Figure 3) a) Cable percussion drilling; b) Rotary drilling



(a)



(b)

MITIGATION THROUGH REDUCTION OF HAZARD

4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE

4.5.1 SMALL AND MEDIUM DIAMETER WELLS - GENERAL ASPECTS

However if the characteristics of the aquifer are known: the permeability influences radius of the cone of depression and the thickness of aquifer, Thiem equations can be used to link the pumping rate to depth of water in the well while pumping (Fig.5a, b). Derivations of the foregoing equations are based on the following simplifying assumptions: 1) uniform hydraulic conductivity within the radius of influence of the well; 2) the aquifer is not stratified, 3) for an unconfined aquifer, the saturated thickness is constant before pumping starts and for a confined aquifer, the aquifer thickness is constant; 4) the pumping well is 100% efficient, that is, the drawdown levels inside and just outside the well bore are at the same elevation and Head losses in the vicinity of the well are minimal; 5) the intake portion of the well penetrates the entire aquifer; 6) the water table or piezometric surface has no slope; 7) laminar flow exists throughout the aquifer and within the radius of influence of the well; 8) the cone of depression has reached equilibrium so that both drawdown and radius of influence of the well do not change with continued pumping at a given rate. For details about Thiem equations see Thiem, 1906 Hydrologische methoden, Leipzig.

Figure 4) Well charateristic curve for a) confined acquifer, b) unconfined acquifer

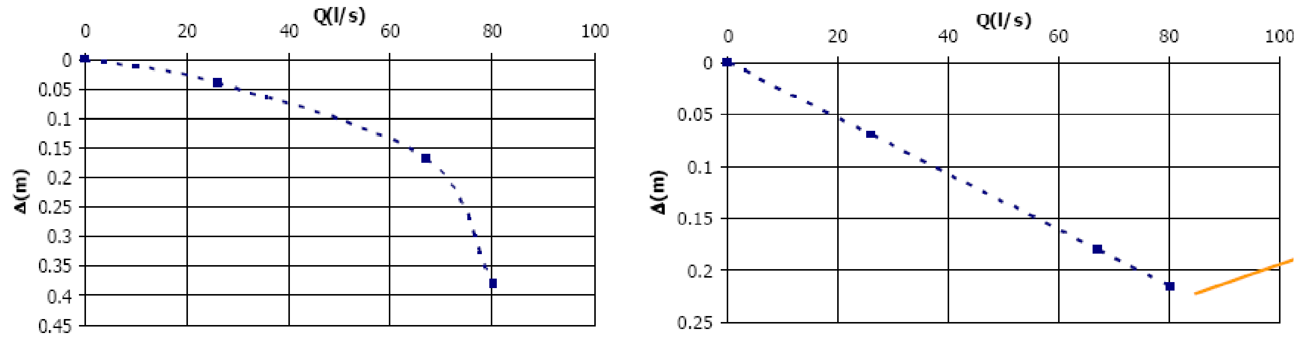
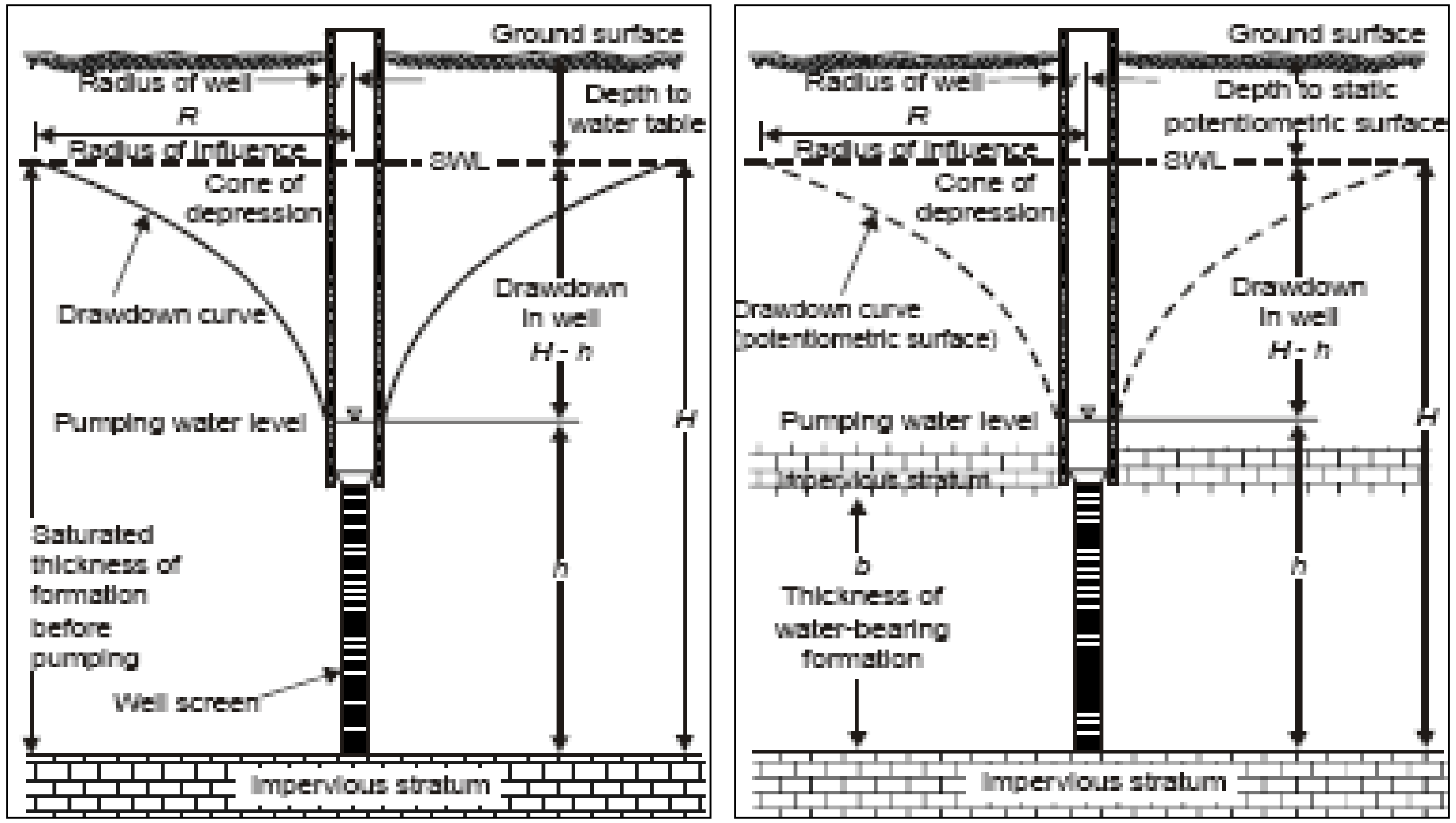


Figure 5) a) Well in an unconfined aquifer and Thiem equation , b) Well in confined acquifer and Thiem equation (Thiem, 1961)

Q = well yield or pumping rate, in m^3/day
K = hydraulic conductivity of the water-bearing formation, in m^3/day/m^2(m/day)
H = static head measured from bottom of aquifer, in m
h = depth of water in the well while pumping, in m
R = radius of the cone of depression, in m
r = radius of the well, in m
h = thickness of aquifer, in m



$$Q = \frac{1.366K(H^2 - h^2)}{\log R / r}$$
(a)

$$Q = \frac{2.73Kb(H - h)}{\log R / r}$$
(b)

MITIGATION THROUGH REDUCTION OF HAZARD

4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE

4.5.1.1 VERTICAL SMALL DIAMETER (<800mm) WELLS –RELIEF OF ARTESIAN PRESSURE

Description

Relief wells, characterized by a small diameter (<800 mm) (Fig.1), may be used to reduce piezometric head in a confined aquifer. No pumping is necessary, relief wells can only discharge water when the piezometric level in the aquifer is above the level of their outlet (Fig. 2left). Therefore a relief well is able only to reduce the piezometric level to the level of the well's outlet. At worst, the level of the outlet may be that of the ground surface, but discharge may also be at lower level, through a pipe installed in a trench (Fig. 2right).

These drainages are mostly appropriate in not very steep slopes where there is not sufficient fall for a gravity drain. Their most frequent application is therefore related to areas downstream of an earth dam or at the toe of a riverbank levee. Therefore in a slope, relief wells may be used to relieve the artesian pressure in a confined aquifer under the toe area, where the ground surface is usually on a flatter gradient (Forrester, 2001).

The technology used to construct the well is discussed in fact sheet 4.5.1. The only thing to add is that the length of the filter might be equal to the thickness of the aquifer.

Design

Once the decrease of the piezometric level is known according to the design, the corresponding discharged flow is calculated. A pipe being able to discharge this flow should be designed.

Figure 1: Classification of wells

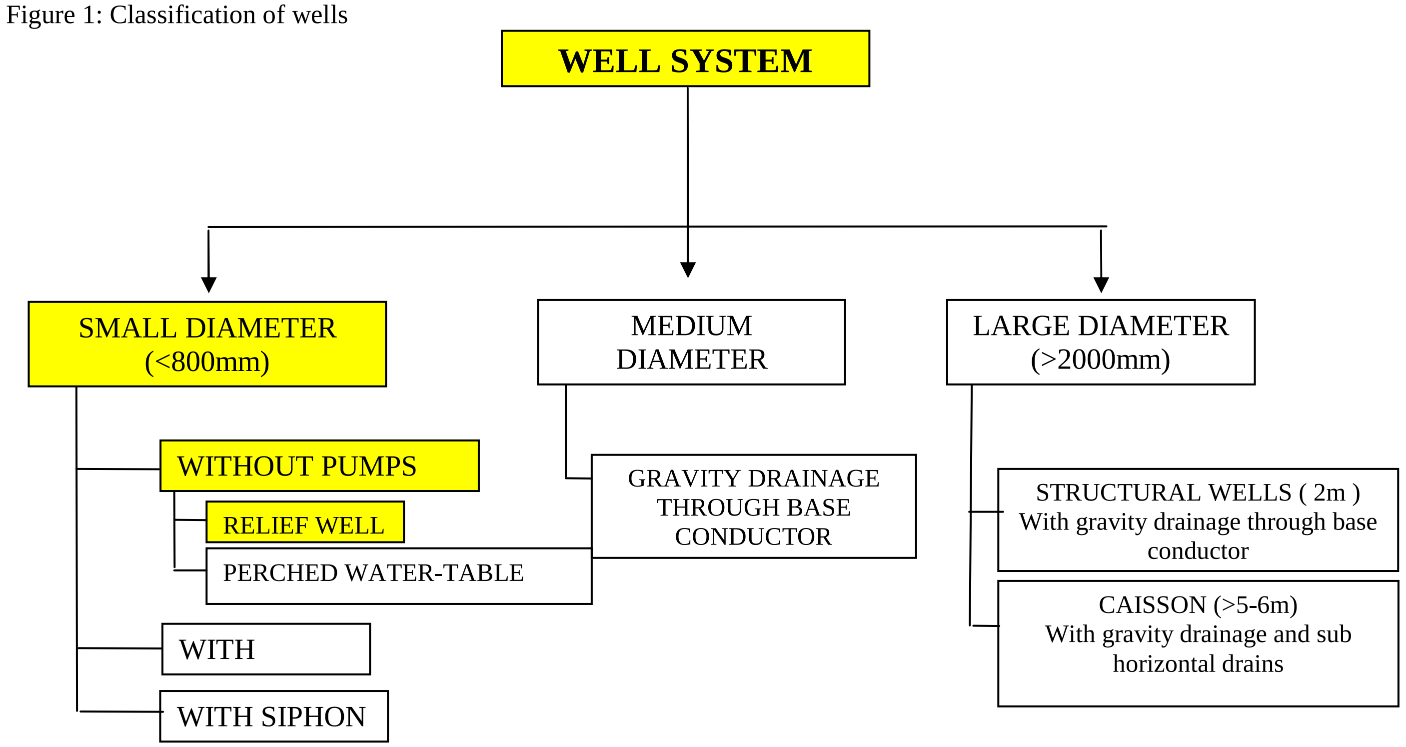
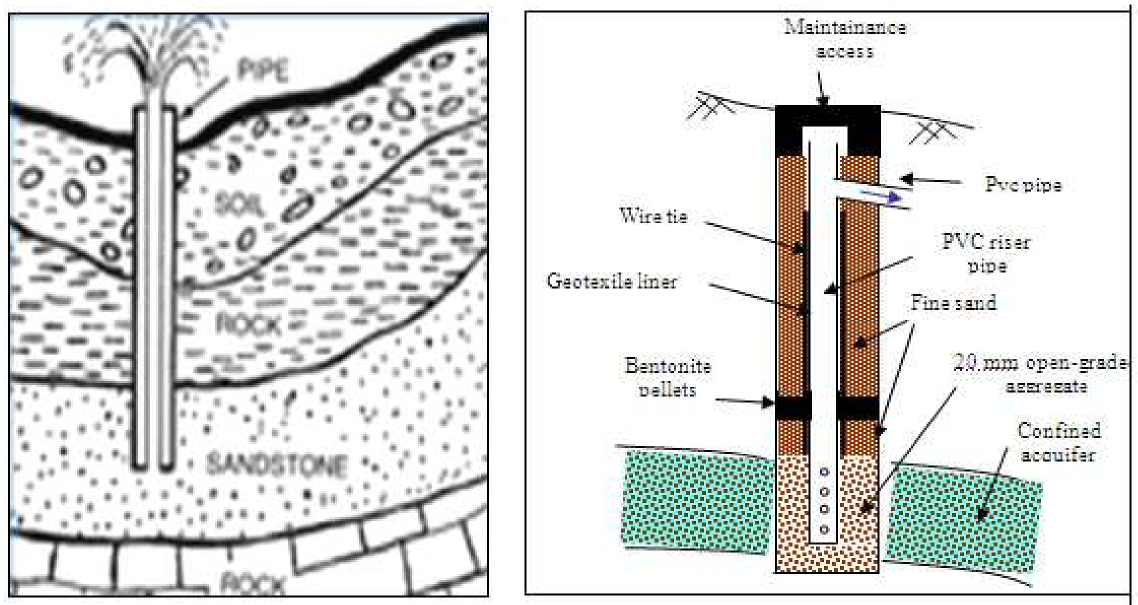


Figure 2: Typical relief well in a confined acquifer with the piezometric level higher (left) and lower (right) than the groundsurface



MITIGATION THROUGH REDUCTION OF HAZARD			
4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE			
4.5.1.1 VERTICAL SMALL DIAMETER (<800mm) WELLS –RELIEF OF ARTESIAN PRESSURE			
APPLICABILITY			
Class	**Descriptor**	**Rating**	**Notes**
Type of movement (Cruden & Varnes, 1996)	Falls	0	A relief well is only able to reduce the piezometric level to the level of the well's outlet. Therefore, relief wells can only discharge water when the piezometric level in the aquifer is above the level of their outlet and only in this case they are used.
	Topples	0	
	Slides	4	
	Spreads	2	
	Flows	4	
Material	Earth	8	
	Debris	4	
	Rock	4	
Depth of movement	Superficial (< 0.5 m)	0	This system can lower the pore water pressure in a confined acquifer and it is usually placed 3-4 m deep from the ground surface.
	Shallow (0.5 to 3 m)	6	
	Medium (3 to 8 m)	8	
	Deep (8 to 15 m)	8	
	Very deep (> 15 m)	6	
Rate of movement (Varnes, 1978)	Moderately to fast	0	The steady-state condition is attained when the hydraulic equilibrium is reached and it is a function of the aquifer properties.
	Slow	4	
	Very slow	8	
	Extremely slow	8	
Groundwater	Artesian	10	This system is suitable only for artesian groundwater.
	High	0	
	Low	0	
	Absent	0	
Surface water	Rain	2	Relief wells modify the piezometric level of the confined aquifer and they are completely separated from the ground surface by means of a grouting cap.
	Snowmelt	2	
	Localized	0	
	Stream	0	
	Torrent	0	
	River	0	
Maturity		8	Technique and design processes are well established and widely used in suitable conditions.
Reliability		7	good working depends strongly on the maintenance.
Implementation		7	
Typical Cost		6	The cost of these drainages is more expensive than the other drainage systems.

Note
Ratings are given on a scale of 1 to 10; the higher the grade, the most suitable is the specific method under consideration to use in landslides of the given characteristics, evaluated individually. Overall suitability to specific case under consideration may be obtained by a weighted average of these ratings, with user defined weights. Zero rating means "not applicable"

MITIGATION THROUGH REDUCTION OF HAZARD

4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE

4.5.1.1 VERTICAL SMALL DIAMETER (<800mm) WELLS –RELIEF OF ARTESIAN PRESSURE

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MITIGATION THROUGH REDUCTION OF HAZARD

4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE

4.5.1.2 VERTICAL SMALL DIAMETER (<800mm) WELLS – UNDERDRAINAGE OF PERCHED ACQUIFER

Description

In a perched water table, water seeping downward is blocked by an impermeable layer of clay or silt, while groundwater saturates the area above the impermeable layer, as shown in Figure 1. An impervious stratum creates a basin that may hold groundwater that is perched above the main water table. A perched water table is not frequent and is well recognizable by geologists and water engineers trough accurate investigations. Perched water is fed by surface water derived from precipitation and snow melt. When the area is urbanized, perched water is further fed by lawn watering, drain from leaking sewer lines, and other man-made sources. A perched water reservoir can be replenished by a water source as far as a mile away (depending also on the involved soils). The size of a perched water reservoir can vary considerably. A small reservoir can pose a seepage problem only after a prolonged wet season, while some perched water reservoirs do not dry up even during dry seasons. However, in the Rocky Mountain region where clay stone bedrock is near the ground surface, the extent of the perched water table can be very extensive.

A small amount of perched water may be drained by drilling holes which cross the impervious basin, therefore small diameter well (<800 mm, see Figure 2) without pumps but with the open bottom into the sand layer placed below the impervious soil layer can be used to under drainage the perched table (Figure 3).

Design

The borehole should be designed to draw down a water flow enough to stabilize the area.

Figure 1: Example of perched table

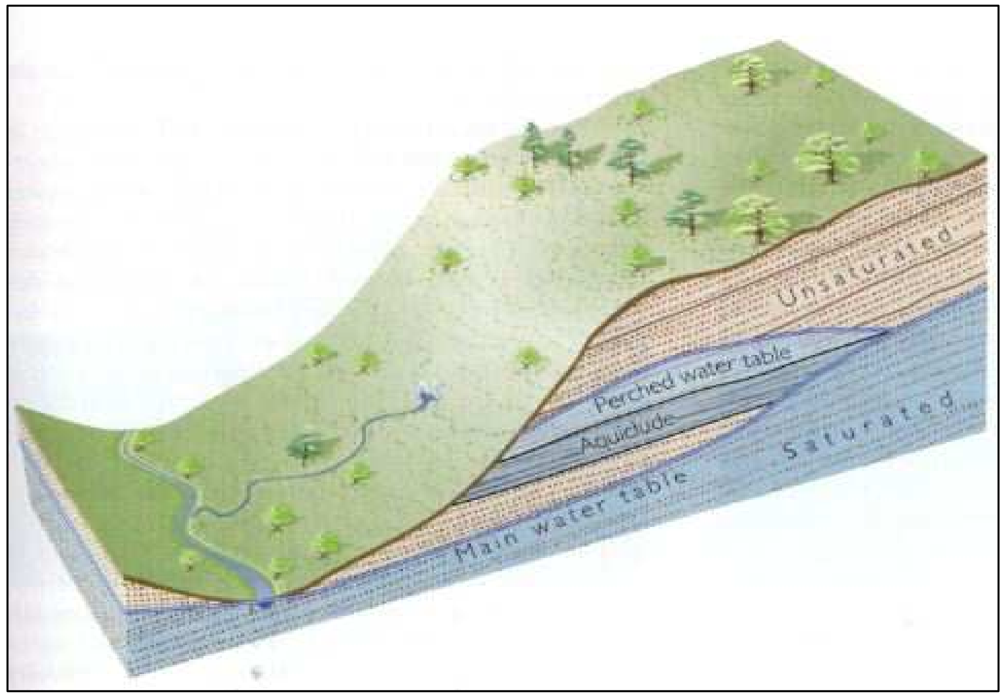


Figure 2: Classification of wells

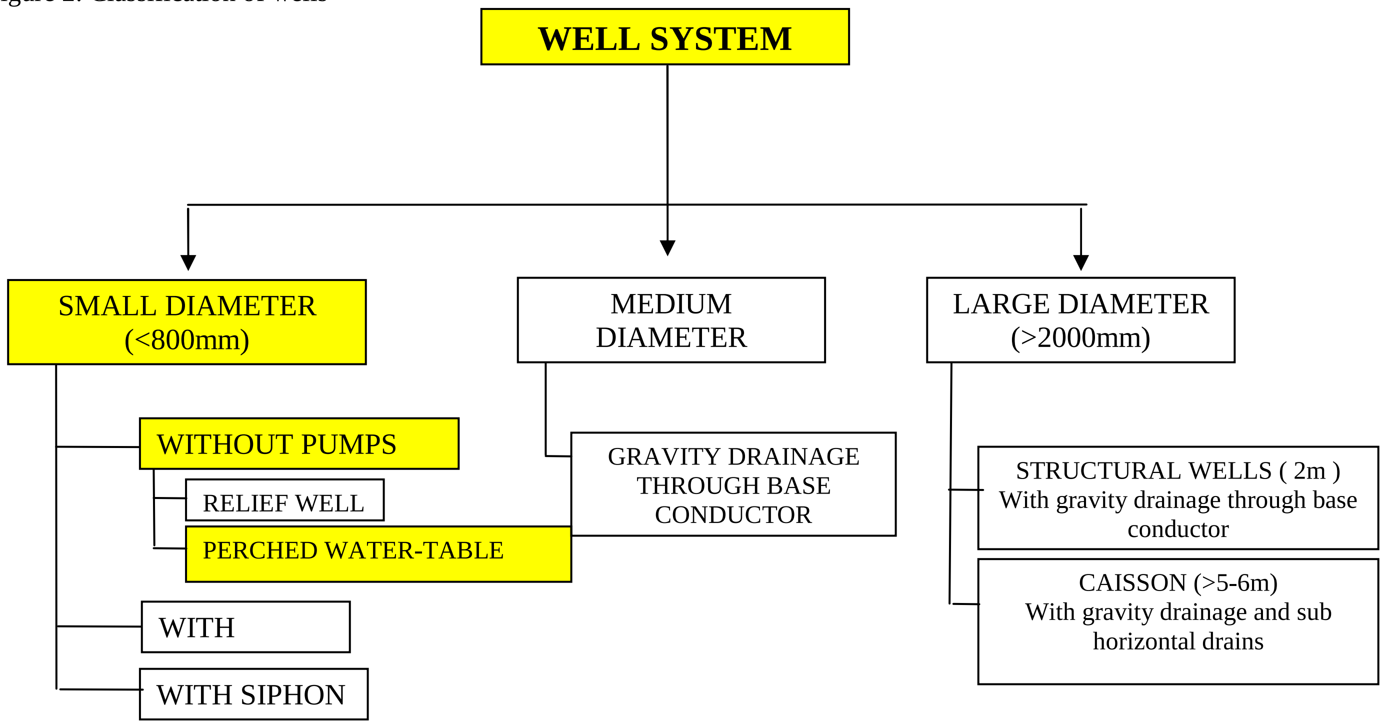
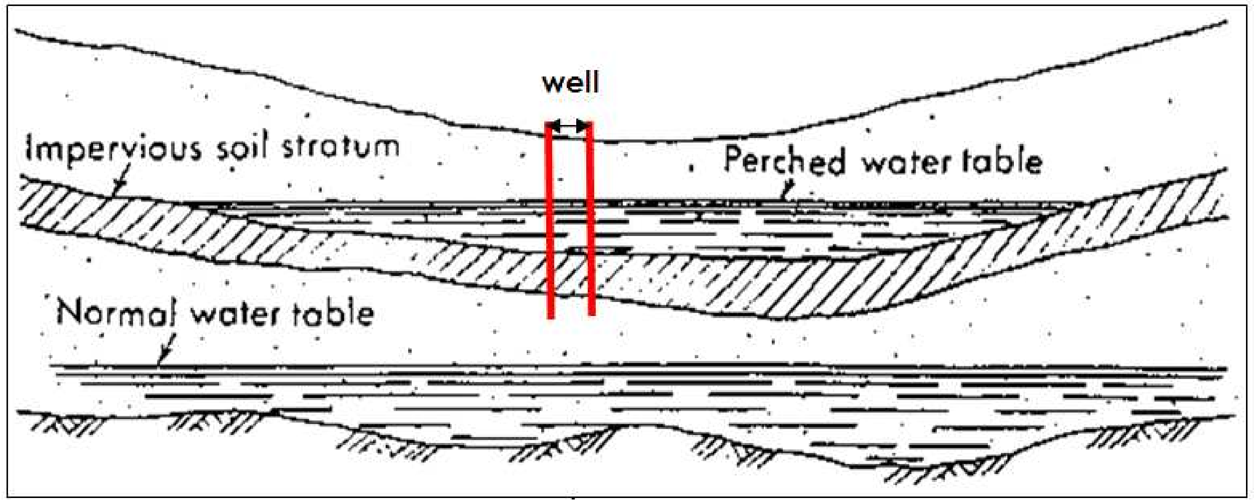


Figure 3: Example of well to underdrain the perched table



MITIGATION THROUGH REDUCTION OF HAZARD

4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE

4.5.1.2 VERTICAL SMALL DIAMETER (<800mm) WELLS – UNDERDRAINAGE OF PERCHED ACQUIFER

APPLICABILITY

Class	Descriptor	Rating	Notes
Type of movement (Cruden & Varnes, 1996)	Falls	2	
	Topples	2	
	Slides	6	
	Spreads	0	
	Flows	0	
Material	Earth	6	The perched water table usually develops into debris layer resting on clay
	Debris	8	
	Rock	4	
Depth of movement	Superficial (< 0.5 m)	0	The effect of lowering of the water table by means of underdrainage is effective of course where the perched basin is placed, usually at 3-8 m deep from the groundsurface.
	Shallow (0.5 to 3 m)	4	
	Medium (3 to 8 m)	6	
	Deep (8 to 15 m)	4	
	Very deep (> 15 m)	4	
Rate of movement (Varnes, 1978)	Moderately to fast	0	The water must have time enough to reach the sand.
	Slow	4	
	Very slow	8	
	Extremely slow	8	
Groundwater	Artesian	0	This system is suitable only for high freatic level.
	High	8	
	Low	0	
	Absent	0	
Surface water	Rain	2	
	Snowmelt	2	
	Localized	0	
	Stream	0	
	Torrent	0	
	River	0	
Maturity		6	
Reliability		6	good working depends strongly on the maintenance.
Implementation		7	
Typical Cost		6	The cost of these drainages is more expensive than the other drainage systems.

Note
Ratings are given on a scale of 1 to 10; the higher the grade, the most suitable is the specific method under consideration to use in landslides of the given characteristics, evaluated individually. Overall suitability to specific case under consideration may be obtained by a weighted average of these ratings, with user defined weights. Zero rating means "not applicable"

MITIGATION THROUGH REDUCTION OF HAZARD
4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE
4.5.1.2 VERTICAL SMALL DIAMETER (<800mm) WELLS – UNDERDRAINAGE OF PERCHED ACQUIFER

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MITIGATION THROUGH REDUCTION OF HAZARD

4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE

4.5.1.3 VERTICAL SMALL DIAMETER (<800mm) WELLS – PUMPS

Description

The role of pumped wells as a mean of slope stabilization is mostly limited to dewatering excavations for structural foundation, where their work is purely temporary. They are not often used as a permanent mean of slope stabilization in fact this technology has been developed mainly in relation to the extraction of groundwater as a resource, but also in relation to structural dewatering problems.

Their main advantages and disadvantages are:

advantages: all types of wells can extract groundwater from locations where gravity methods are impractical. Their drainage capacity can be increased at any time by placing more wells. In the case of pumped wells, drainage performance may also be adjusted by altering the on/off switching levels, by increasing or decreasing the pumping rate, or hutting some pumps down (Forrester, 2000);

disadvantages: pumped wells require an on-going commitment for maintenance and intermittent or continuous operations. They are therefore only used if stabilization by drainage is essential, but no method of gravity drainage is feasible. Therefore the maintenance costs very much and influences the service-life and the good working of the wells.

Pumping system

The selection of the pumping plant will be influenced by the quantity of water to be extracted and the height to which it must be pumped to ground level. Typical details of three pumping systems that are most commonly used for dewatering, and consequently for slope stabilization, are:

Wellpoint (Fig.2a, 3a, 4a): this consists of a well screen set on the end of a 38 mm diasteel pipe. Several wellpoints are connected to a common pump through a header pipe at ground level. A wellpoint may be driven into the ground or placed in a borehole, but it is usually jetted into place to the required level. This is done by applying water pressure to the tip through a temporary jetting pipe, with a rubber ball valve that allows a jet of water to be directed downwards. The valve closes when the jetting pipe is removed and the direction of flow is reversed for groundwater extraction. The wellpoint's biggest disadvantage is that it works by suction and is therefore unable to raise water more than about 7.5 m. The maximum limit of drawdown: 3-4 m in silty fine sands, 5-5.5 m generally. It's common practice to use two pumps initially and then continue using one pump at a time with the second one available as a stand-by.

Ejector (Fig.2b, 3b, 4b): also known as an eductor, this is placed in a cased borehole with a well screen as part of the casing. Its essential futures are a jet-the supply water-directed upwards through a venture. The venture is also open to the surrounding groundwater that has passed through the screen and into the casing. This water is carried with the supply water through the venture to the ground level. There, discharge in excess of the supply water flow is wasted. There are two different pipe arrangements. The first uses two pipes - one to lead the supply water to the ejector and the other to carry up the combined supply water and the groundwater. The other arrangement requires only a single pipe. The disvantages of an ejector are its high power consumption, since the same flow of supply water must be pumped continuously out of well for as long as pumping continues. Economically, it is not worth using it to raise water more than about 40 m. The advantage of the eductor system is that the water table can be lowered in one stage from depths of 10-45 m. However the efficiency of such system is lower than that of other pumping system. They become economically competitive in soils of relativity low permeability. Well diameter: 50 mm minimum-Well depth: up to 30 m.

Submersible pump (Fig. 2c, 3c, 4c): This type also required a cased boreholes with a well screen as part of the casing. The lowest component of the pump is an electric motor connected by a cable to the power source at ground level. Above the motor are the water inlet and pump screen, and the several pump rotors, one above the other. To prevent the motor from overheating, water must be kept moving past it while it is operating. Therefore, a switch that turns off the power at a lower water level is required. Submersible pumps are available in a wide variety of sizes and capacities. Switching is controlled by electrical contacts, similar in principle to those used to monitor waste levels in standpipe piezometers. A remote alarm system, also required for each pump, warns of pump malfunction or failure. The submersible pump is suitable for deep wells. Bore diameter: 150 mm to 500 mm - Well depth: up to 150 m.

However in Figure 5 the application field of this pumping system is shown as a function of soil permeability and design drawdown

Figure 1: Classification of wells

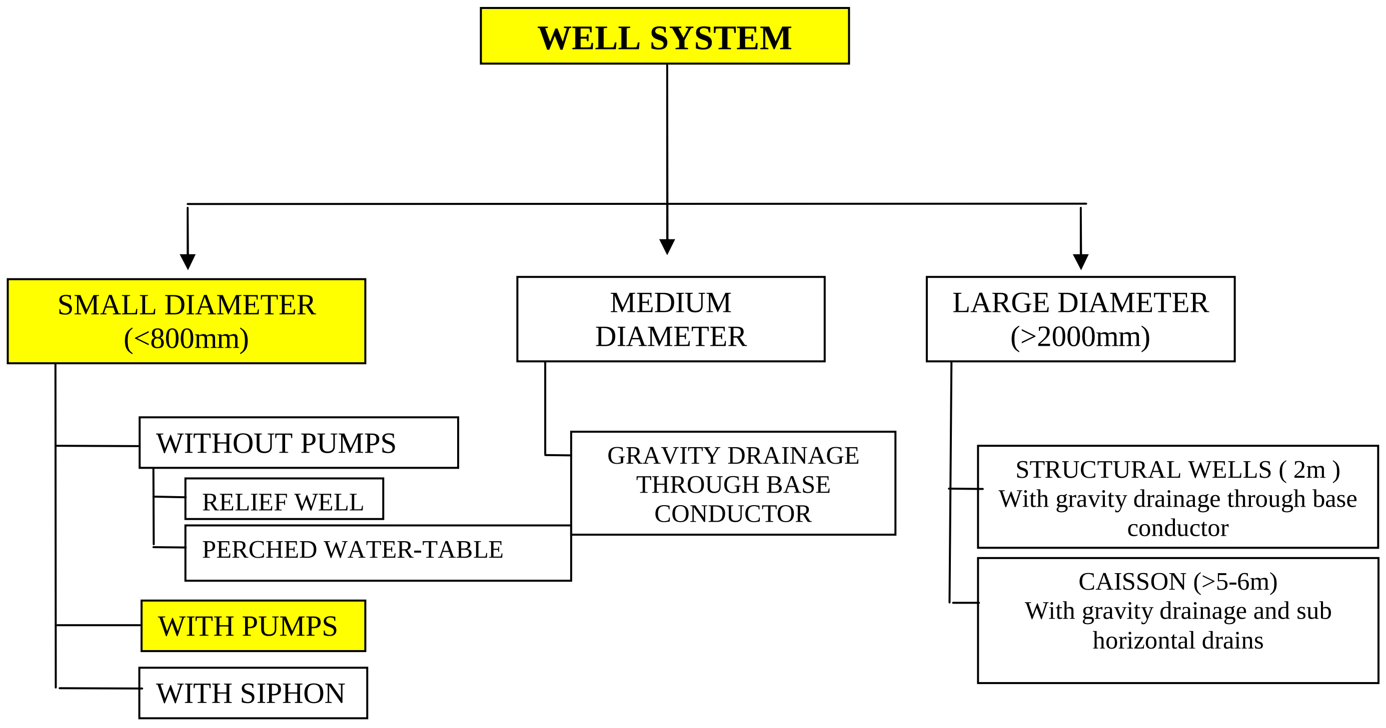
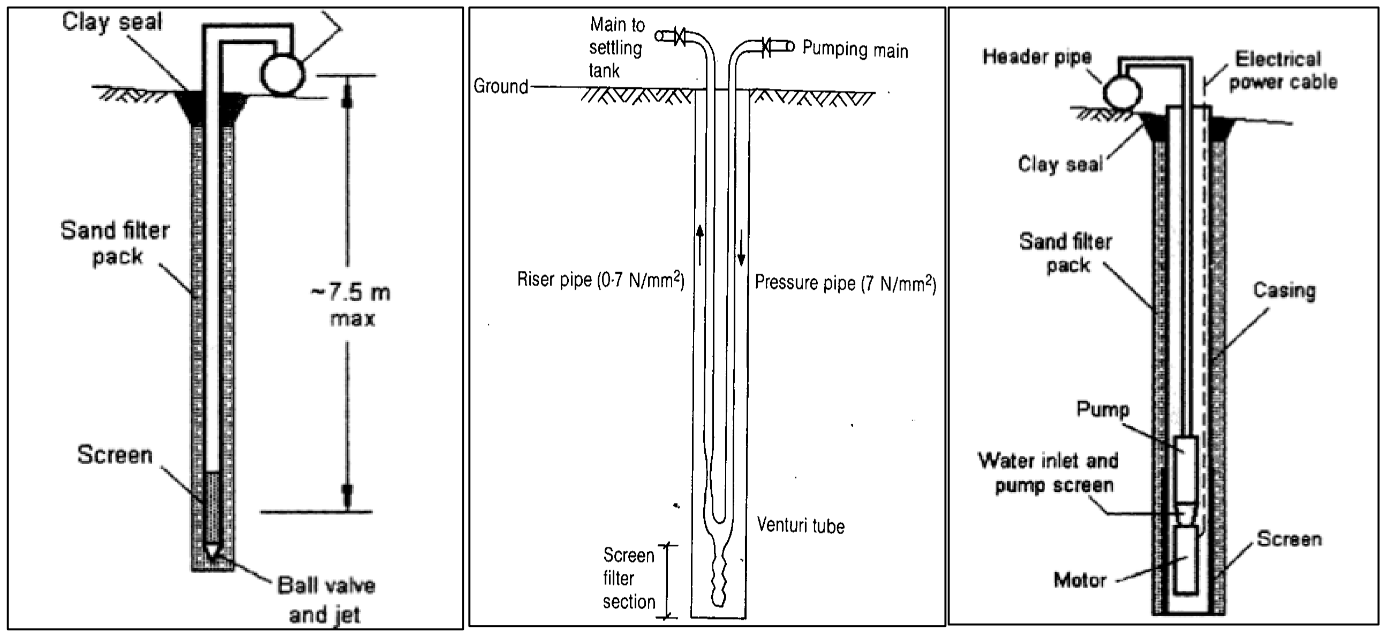


Figure 2: Wells with different pumping systems: a) wellpoints; b) ejector; c) submersible pump.



(a) (b) (c)

MITIGATION THROUGH REDUCTION OF HAZARD

4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE

4.5.1.3 VERTICAL SMALL DIAMETER (<800mm) WELLS-PUMPS

Design

Each pump system can work up to a maximum depth. Wells are uneconomic compared to other drainage systems, especially when shallow depth must be reached; therefore well points are usually not adopted for slope stabilization. Deep wells (around 30 m) with submersible pumps are the most common system.

The type of pump to be adopted is a function of the pumping flow. Having determined how much the piezometric level must be lowered to obtain the required condition of slope stability, the flow is determined by means of the characteristic curve of the well (see fact sheet 4.5) or the Thiem equation. All suppliers provide the characteristic curve of their pumps to choose the most appropriate model and the optimum working point according to the design parameters. Typical submersible pump-characteristic curves are shown in the figure 6, for each pump type (motor type) at assigned diameter.

Figure 5: Application field of different type of pumping systems as a function of soil permeability and design drawdown.

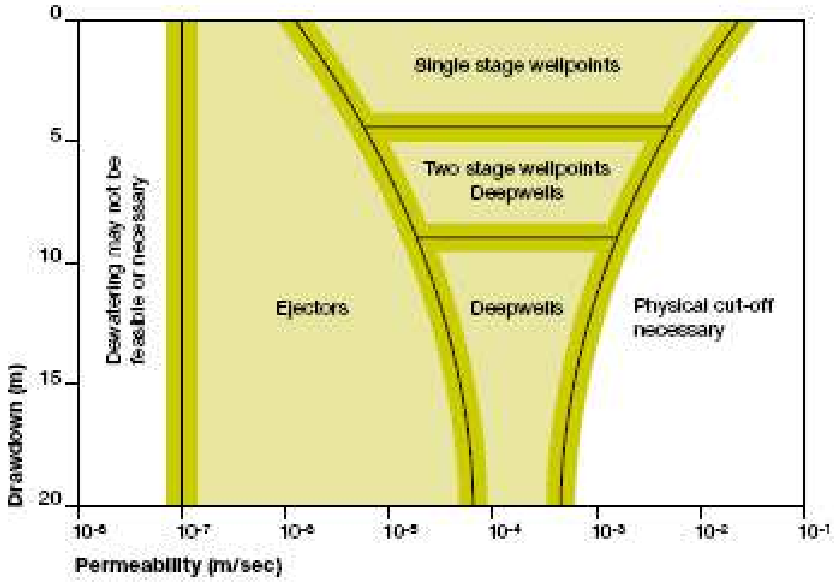


Figure 6: Charcteristic curves of submersible pumps.

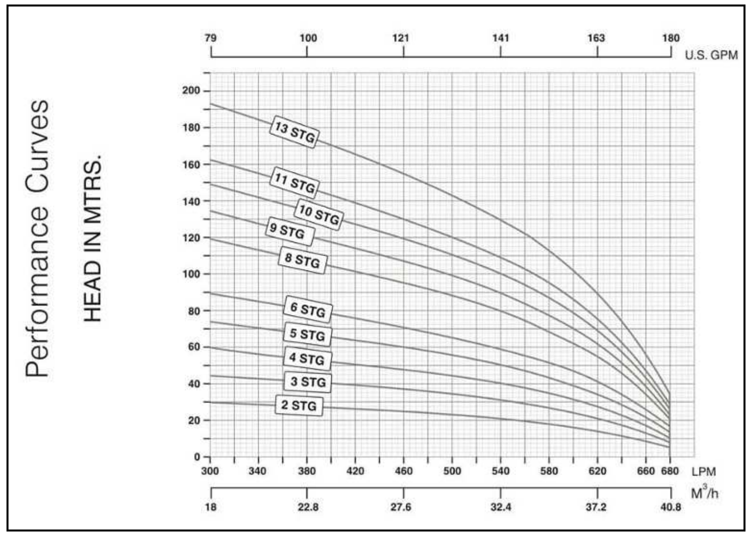


Figure 3: a) Pumps for well system, b) eductor pump, c) submersible pump.

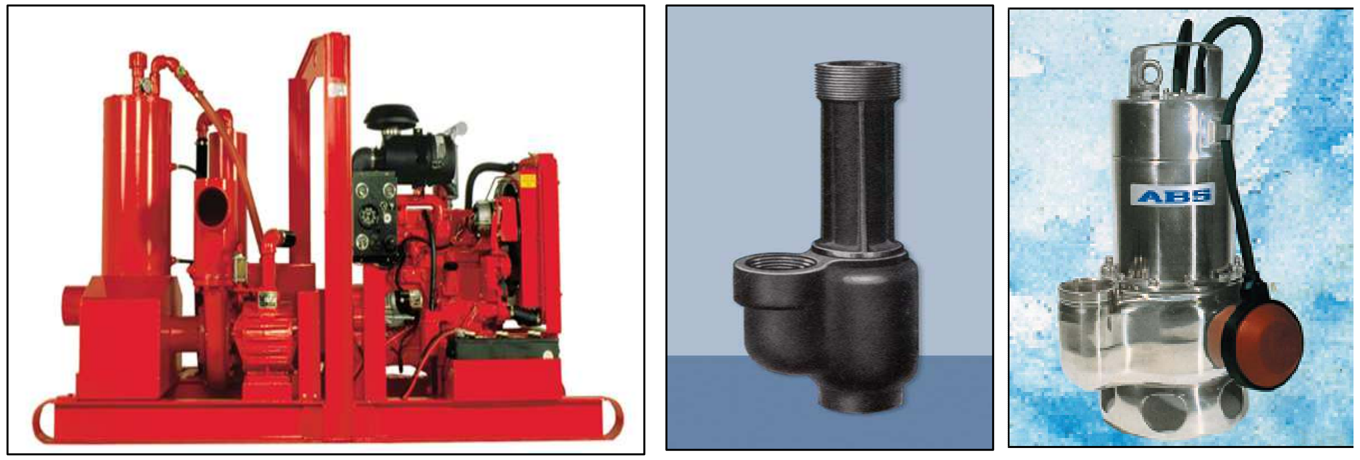


Figure 4: Examples of a) well point system, b) educator system, c) submersible pump system

a) b)

c)

MITIGATION THROUGH REDUCTION OF HAZARD

4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE

4.5.1.3 VERTICAL SMALL DIAMETER (<800mm) WELLS-PUMPS

APPLICABILITY

Class	Descriptor	Rating	Notes
Type of movement (Cruden & Varnes, 1996)	Falls	0	The role of pumped wells as a means of slope stabilization is mostly limited to dewatering excavations for structural foundation, or for temporary drainage of large landslides while awaiting construction of drainage adits. Hence their function is purely temporary. They are not often used as a permanent means of slope stabilization.
	Topples	0	
	Slides	5	
	Spreads	3	
	Flows	0	
Material	Earth	5	Deep well systems are effective in a range of soil conditions from gravel to silty fine sands
	Debris	6	
	Rock	4	
Depth of movement	Superficial (< 0.5 m)	0	Deep wells are suitable for very deep slip surface up to 30 m..
	Shallow (0.5 to 3 m)	0	
	Medium (3 to 8 m)	5	
	Deep (8 to 15 m)	8	
	Very deep (> 15 m)	8	
Rate of movement (Varnes, 1978)	Moderately to fast	0	The steady-state condition is attained when the cone of depression reaches the equilibrium; time necessary is a function of the aquifer properties.
	Slow	2	
	Very slow	8	
	Extremely slow	8	
Groundwater	Artesian	6	suitable for high freatic water-table, in particular centrifugal pumps could be used instead of submersible ones when the heights to be overcome are less than 5-6 m.
	High	8	
	Low	6	
	Absent	0	
Surface water	Rain	2	Not suitable to drainage shallow water.
	Snowmelt	2	
	Localized	0	
	Stream	0	
	Torrent	0	
	River	0	
Maturity		7	Technique and design process are well established and widely used in suitable conditions
Reliability		6	Reliability in the long term depends on maintenance, especially of the pump system. Difficult to predicting actual drawdown pattern in complex soil.
Implementation		7	Rapid installation, especially well points. Many experienced suppliers available. Deep wells require good working platform for drilling rig
Typical Cost		5	Deep wells are relatively expensive depending on the number installed, depth and strata. Eductor is the most expensive system.

Note
Ratings are given on a scale of 1 to 10; the higher the grade, the most suitable is the specific method under consideration to use in landslides of the given characteristics, evaluated individually. Overall suitability to specific case under consideration may be obtained by a weighted average of these ratings, with user defined weights. Zero rating means "not applicable"

MITIGATION THROUGH REDUCTION OF HAZARD

4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE

4.5.1.3 VERTICAL SMALL DIAMETER (<800mm) WELLS – PUMPS

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WJ Groundwater Ltd. Slope stability – Well points system.

MITIGATION THROUGH REDUCTION OF HAZARD

4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE

4.5.1.4 VERTICAL SMALL DIAMETER (<800 mm) WELLS – SIPHONING

Description

This technique was conceived and developed in France. It consists of isolated drainage wells of diameter 100 to 300 mm, equipped with a slotted PVC or a perforated or micro perforated steel pipe, with the annulus between pipe and soil filled with draining material, as explained in fact sheet 4.5.. The wells are pumped using a siphon driven by the fall in elevation of the slope (Fig. 2), overcoming the inconvenience of installing and operating a pump in each well.
Its use in slope stability is restricted for two reasons (Forrester, 2000):

6. A siphon has a maximum theoretical lift of 10.2 m (equivalent to atmospheric pressure); however, it has a maximum practical lift of 8.3 m due to the vapor pressure of water and friction head loss.
7. If a sufficient air enters the siphon at any time, the pumping is broken. Flow can only resume if priming is restored.

A siphon is a familiar device for moving water from one level to a lower one; in its simplest form this consists of an inverted U-tube, both legs being full of water, and the flow is generally calculated by equating the total head producing flow, i.e. the difference of heads in the two reservoirs, h, to the sum of the frictional and other losses in the pipe and of the velocity head produced (for details see Citrini, Noseda 1986). System flow would decrease as h decreases due to drawdown in the well. Equilibrium would occur at the drawdown, yielding the system flow capacity.
Siphons require priming (initial filling of line) to initiate flow. After priming, the siphon will passively convey liquid from the point of higher hydraulic head to the one of lower head indefinitely so long as the head differential is maintained and the prime is not lost. For flow tio be maintained, it is necessary that at all times:

- inlets and outlets are submerged, to prevent air from being drawn into the siphon line,
- gases which tend to accumulate in the siphon line as they come out of soultion due to the sub-atmospheric pressures. are removed

In fact, as the summit (minimum) pressure decreases, dissolved gases in the groundwater come out of solution and help form intermittent discontinuities as the pressure approaches a true vacuum. A break in the siphoning action occurs at a point less than the theoretical limit as the summit pressure continues to decrease.
One or both of the following methods may be used to remove the gases which have degassed from the liquid, thus maintaining full siphon flow:

- Maintenance of the minimum flushing velocity required to transport gases out to the end of the siphon.
- Use of air chambers at the siphon crest This makes the system less than entirely passive, since the chambers require periodic recharging.

Management of gas within the siphon line is considered to be of the greatest importance in the maintenance of siphon flow. Gas bubble transport, accumulation, agglomeration, and entrapment are controlled by fluid flow velocity, gas buoyancy, and siphon line grades and inside diameter discontinuities (i.e. fittings). Gas bubble transport in the upward leg of the siphon line is facilitated by higher fluid flow velocities, by a continuous upward siphon line grade (no localized high points), and the minimization or elimination of fittings which produce discontinuities in the internal diameter of the siphon line. The direction of gas bubble transport, if any, in the siphon line downward leg is determined by whether transport due to fluid flow velocity or gas buoyancy is dominant. In order to utilize the minimum flushing velocity to maintain full flow in the siphon line downward leg, the fluid flow velocity must be dominant in the downward leg. Additionally a continuous, downward, siphon line, grade (i.e. no localized high points) and the minimization or elimination of fittings which produce discontinuities in the internal diameter of the siphon line, is necessary.

Design

The siphon system is a very effective solution to slope stability problems in terms of adaptability and durability. The water table can be lowered to 8.5 m vertically below the surface when the suction inlet is placed at 10 m below the crown of the siphon. Depending on the gradient of the slope, it is possible to achieve greater effective lowering of the water table if the length and the slope of the wells are modified. Both the diameter and the number of siphon pipes depend on the drainage flow. Diameters range from 10 mm for 150 litres/hour per well, to 25 mm for 1 m3/hour per well. This system proves to be economically advantageous and relatively simple to set up even if it necessitates a programme of controls and maintenance.

Figure 1: Classification of wells

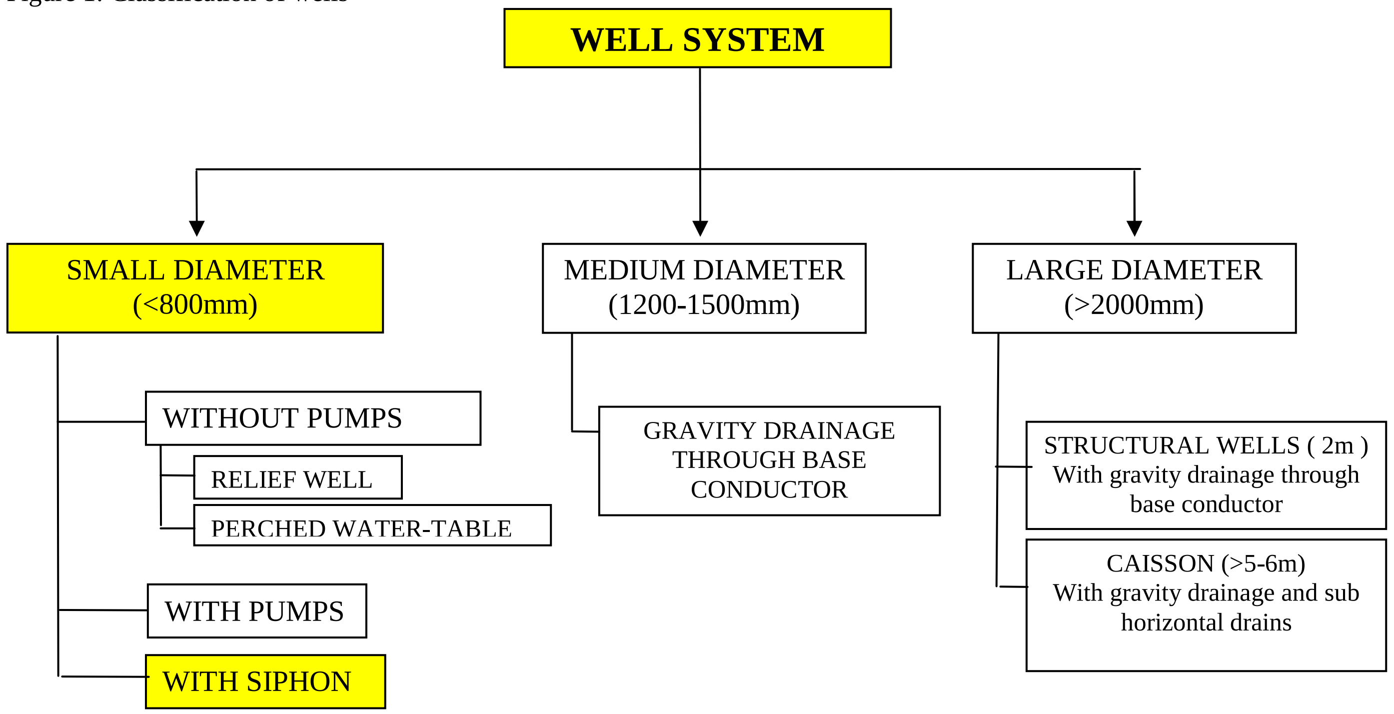
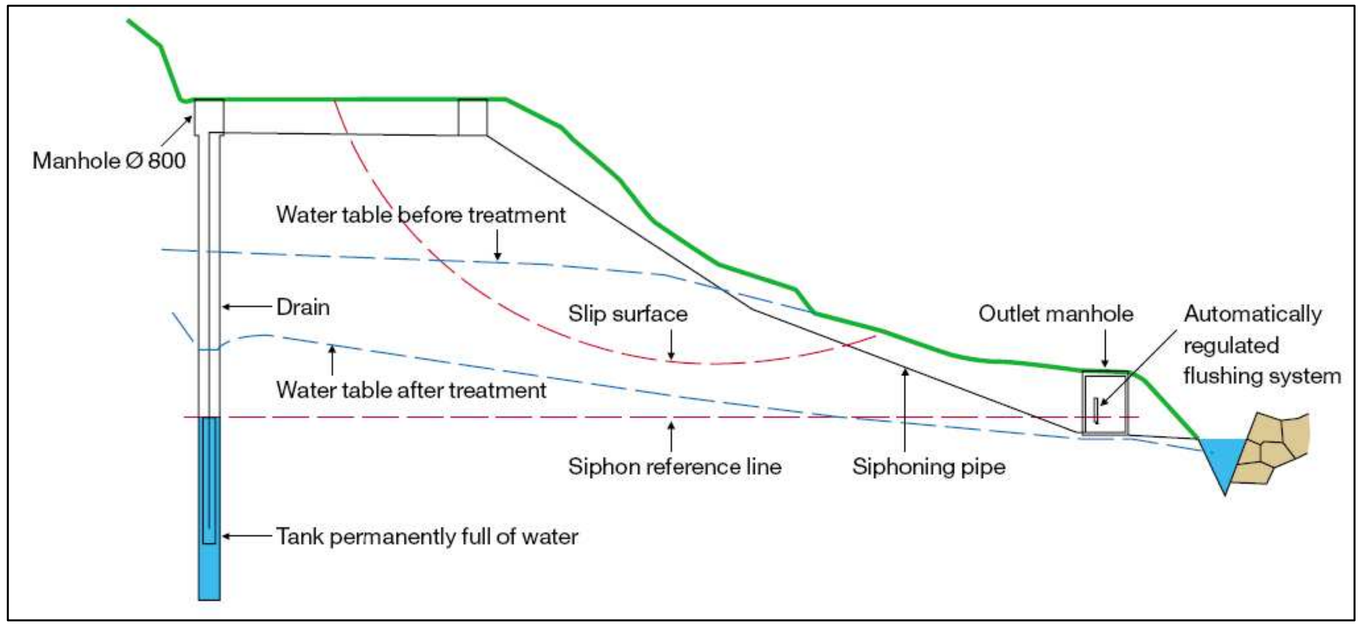


Figure 2: Section showing well and siphon installation (from WJ Groundwater Ltd)



MITIGATION THROUGH REDUCTION OF HAZARD

4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE

4.5.1.4 VERTICAL SMALL DIAMETER (<800 mm) WELLS – SIPHONING

Figure 3: Outlet manhole with flushing system installed

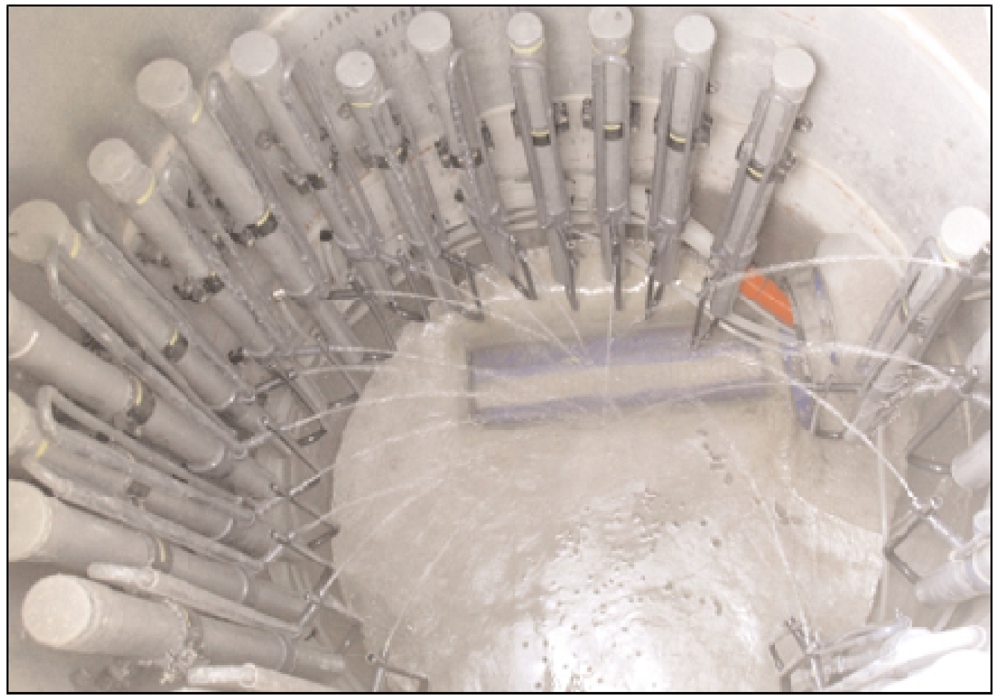
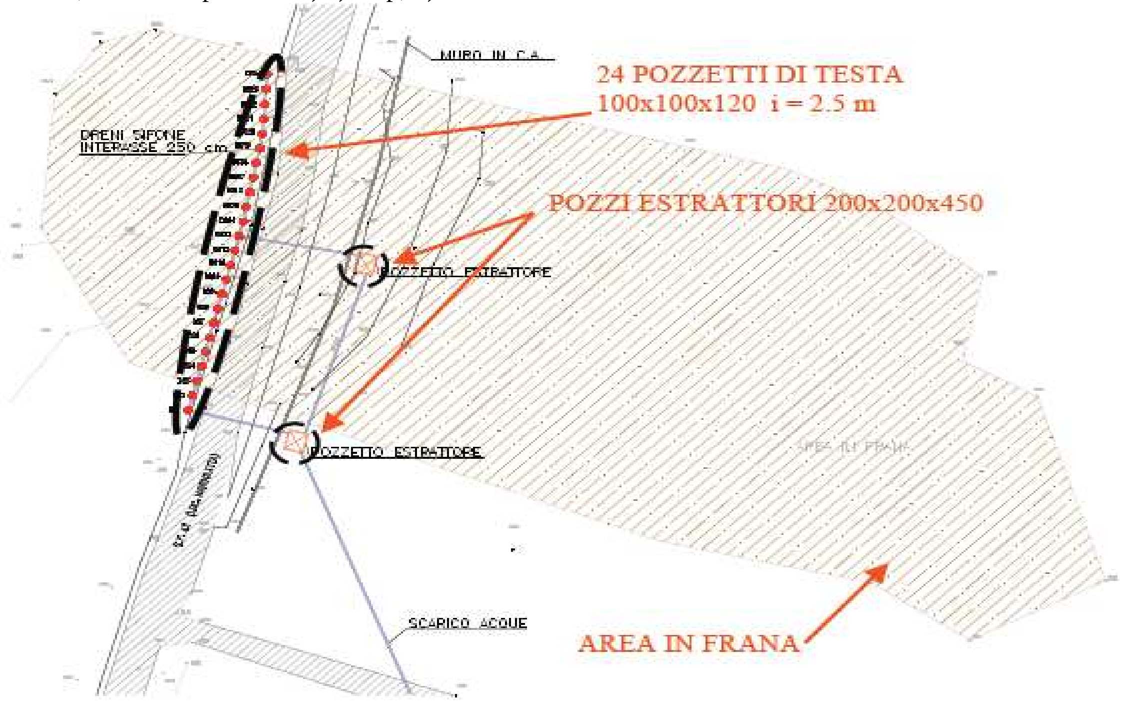


Figure 4: a) Head well; b) Well manhole with duct for siphon tubes

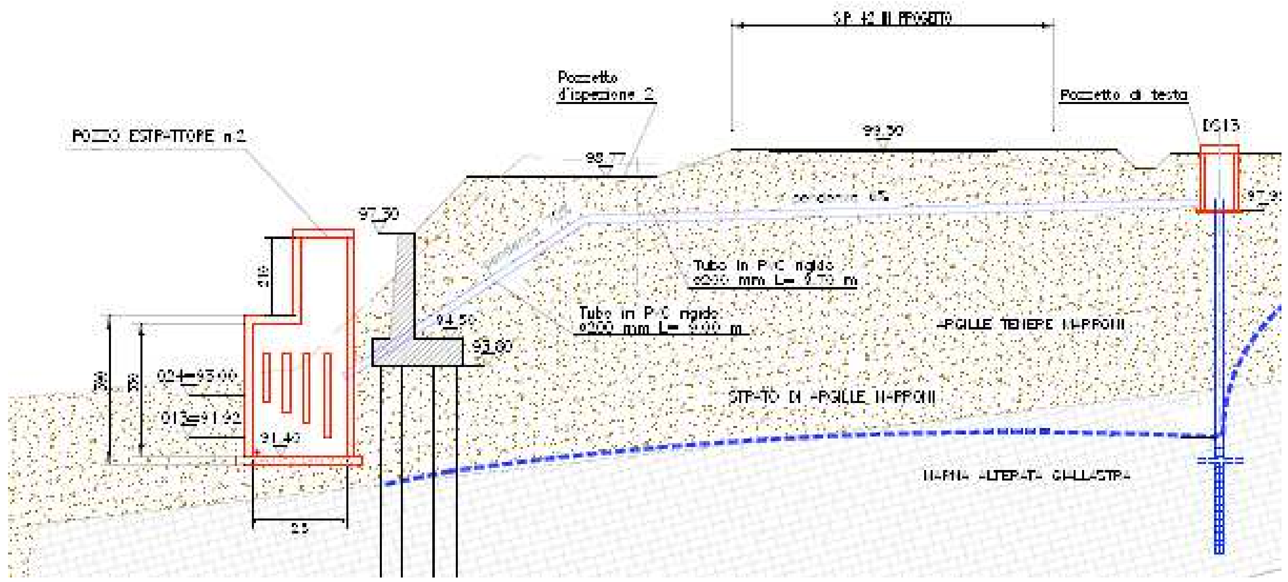
a) 

b) 

Figure 5: Well - Siphon system executed by G.E.AS company in Casale Monferrato (Alessandria) (from I sistemi drenanti nei dissesti del territorio, Torino 7 aprile 2006) a) map; b) transversal section



(a)



(b)

MITIGATION THROUGH REDUCTION OF HAZARD			
4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE			
4.5.1. 4 VERTICAL SMALL DIAMETER (<800 mm) WELLS – SIPHONING			
APPLICABILITY			
Class	**Descriptor**	**Rating**	**Notes**
Type of movement (Cruden & Varnes, 1996)	Falls	0	This system usually is adopted to stabilize landslides characterized by a circular surface. Depending on the gradient of the slope, it is possible to achieve greater effective as regards the lowering of the water table if the length and the slope of the wells are adjusted, taking into account each situation.
	Topples	0	
	Slides	6	
	Spreads	2	
	Flows	0	
Material	Earth	6	
	Debris	6	
	Rock	4	
Depth of movement	Superficial (< 0.5 m)	0	The siphon can lower the water table to 8.5 - 9 m below ground level, thus it can be most effective for slip surfaces up to 10-11 m.. deep
	Shallow (0.5 to 3 m)	4	
	Medium (3 to 8 m)	6	
	Deep (8 to 15 m)	8	
	Very deep (> 15 m)	4	
Rate of movement (Varnes, 1978)	Moderately to fast	0	The steady-state condition is attained when the cone of depression reaches the equilibrium; this time is a function of the acquifer properties
	Slow	2	
	Very slow	8	
	Extremely slow	8	
Groundwater	Artesian	6	This system is suitable for shallow freatic water-table
	High	8	
	Low	6	
	Absent	0	
Surface water	Rain	2	Not suitable to drain shallow water.
	Snowmelt	2	
	Localized	0	
	Stream	0	
	Torrent	0	
	River	0	
Maturity		5	These technique and design processes are used especially in France, at least in Italy and in U.K
Reliability		6	The siphon system is a very effective solution to slope stability problems in terms of adaptability and durability. The good working depends strongly on the maintenance in particular by the management of gas inside pipes..
Implementation		7	System implementation is easy, especially in absence of pumps.
Typical Cost		5	Costs depend on the maintenance

Note
Ratings are given on a scale of 1 to 10; the higher the grade, the most suitable is the specific method under consideration to use in landslides of the given characteristics, evaluated individually. Overall suitability to specific case under consideration may be obtained by a weighted average of these ratings, with user defined weights. Zero rating means "not applicable"

MITIGATION THROUGH REDUCTION OF HAZARD

4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE

4.5.1.4 VERTICAL SMALL DIAMETER (<800 mm) WELLS – SIPHONING

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WJ Groundwater Ltd. Slope stability - Siphon drain system: Permanent gravity-fed siphon drains®.

MITIGATION THROUGH REDUCTION OF HAZARD

4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE

4.5.2 VERTICAL MEDIUM DIAMETER (1200-1500mm) WELLS- GRAVITY DRAINAGE THROUGH BASE CONDUCTOR

Description

This technique consists of forming a deep drainage screen in low permeability soils by installing alignments of wells at 6 to 8 m spacing, connected at the base by drainage pipes to allow the gravity discharge of the water collected in the wells (Figure 1, Leoni et al. 2003). A typical plan and longitudinal section is shown in Figure 2.

The diameter of the wells is typically 1200 to 1500 mm (scheme of Figure 3). They can reach typical depths of 20 to 30 m and in particular cases more than 50 m (Beer et al, 1992, Manassero, 2001). They are excavated using the same equipment and techniques used for bored piles without bentonite mud (Figure 4).

The wells are typically of two types:

Standard wells are filled with drainage material, simultaneously extracting the casing used for temporary support of the hole during drilling. The top of the well is sealed with say minimum 1.0m of impervious fill and topsoil to prevent infiltration of surface runoff.

Inspection wells are formed by installing in the well a permanent 1200 mm diameter corrugated hot galvanized steel casing perforated near the base (Figure 5), filling the annular space between the casing and the borehole with drainage material while extracting the temporary casing as above. These wells are placed at suitable distance along the array, typically one every three wells (Figure 3). Besides being used to drill the base conductor, inspection wells are used to monitor the correct performance of the system and in particular to measure and, if necessary, to regulate the flow rate.

The base conductor, which allows the wells to discharge by gravity, is the main feature of this technology. It typically consists of twin pipes (to guarantee adequate redundancy), installed by drilling through the casings from one inspection well to the other by means of mini-probes (Figures 9 and 10) and installing the pipe in short (450 mm) sections. Inspectionable wells are completed with access ladders, head and bottom sealing and the installation of manhole covers in reinforced concrete.The typical detail of inspection wells is shown in Figure 6. Typical applications are shown in Figures 7 and 8.

Increasingly, the focus on safety of construction and ever greater restrictions on working pracices tend to make the traditional method of forming the base conductor impractical, since it requires man entry to the base of the well. This may be obviated in whole or in part by the use of directional drilling**.**

Design

The depth of the wells and the minimum section of the base conductor are determined by conventional hydraulic calculations based on the required drawdown and the associated flow. Spare capacity should be provided, to minimize maintenance requirements.

Figure 1: Schematic longitudinal section of an array of medium diameter wells (source. Leoni et al. 2003)

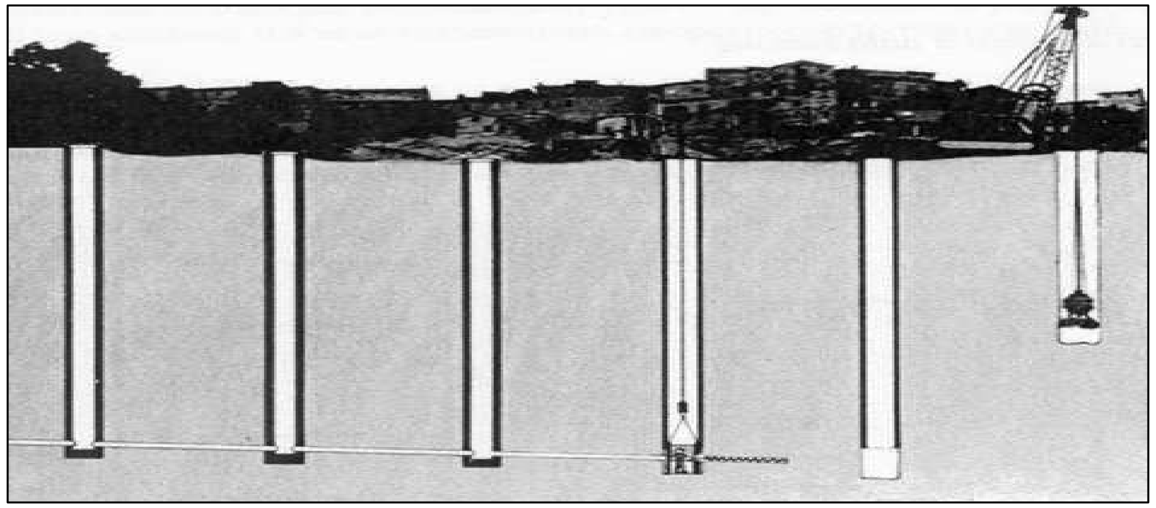


Figure 2: Typical plan and longitudinal section (source: SGI-MI project files)

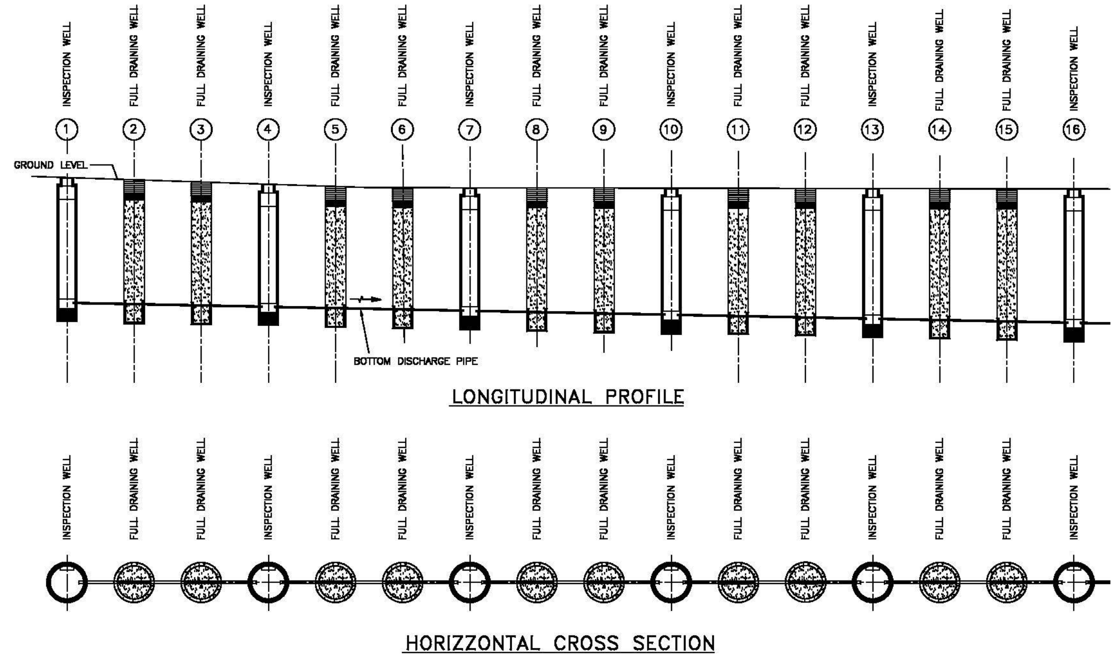
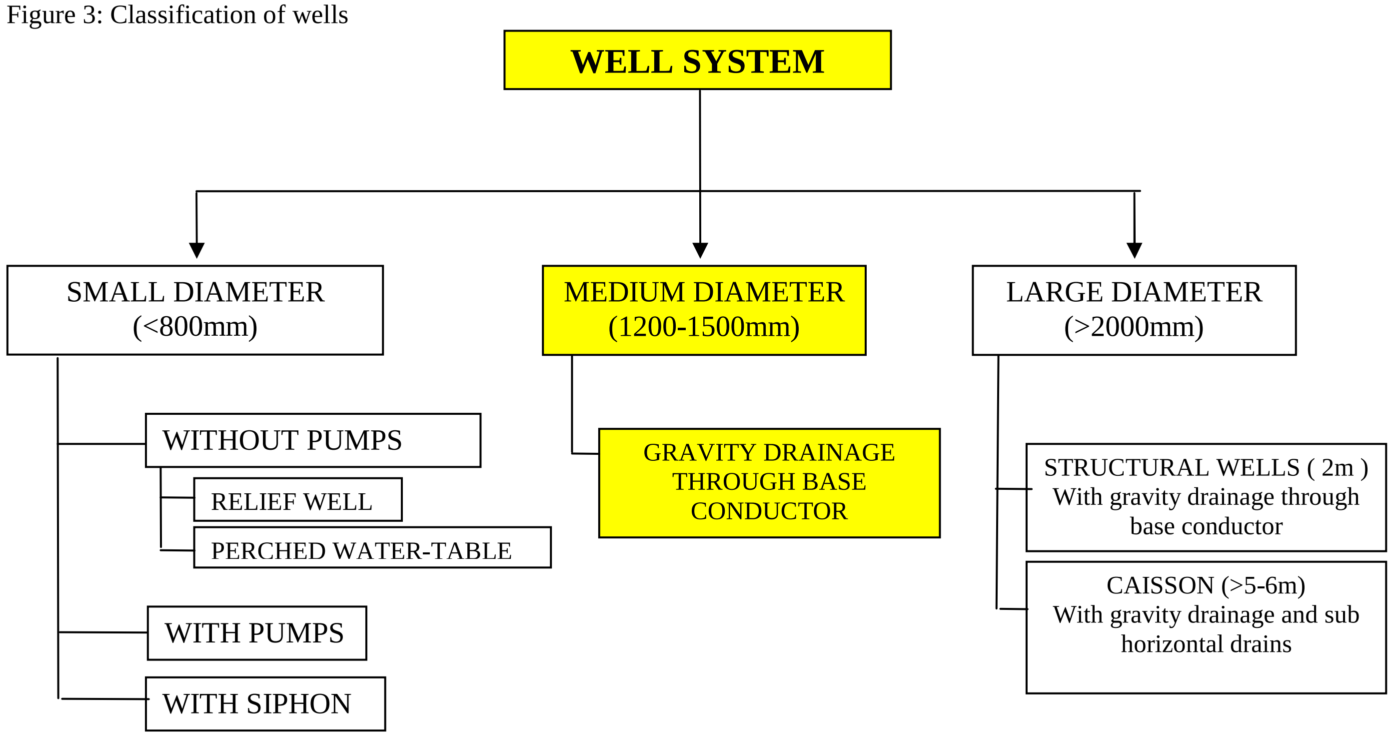


Figure 3: Classification of wells



MITIGATION THROUGH REDUCTION OF HAZARD

4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE

4.5.2 VERTICAL MEDIUM DIAMETER (1200-1500mm) WELLS- GRAVITY DRAINAGE THROUGH BASE CONDUCTOR

Figure 4: Drilling equipment

Figure 5: installation of permanent casing in inspection wells

Figure 6: Typical detail of inspection well

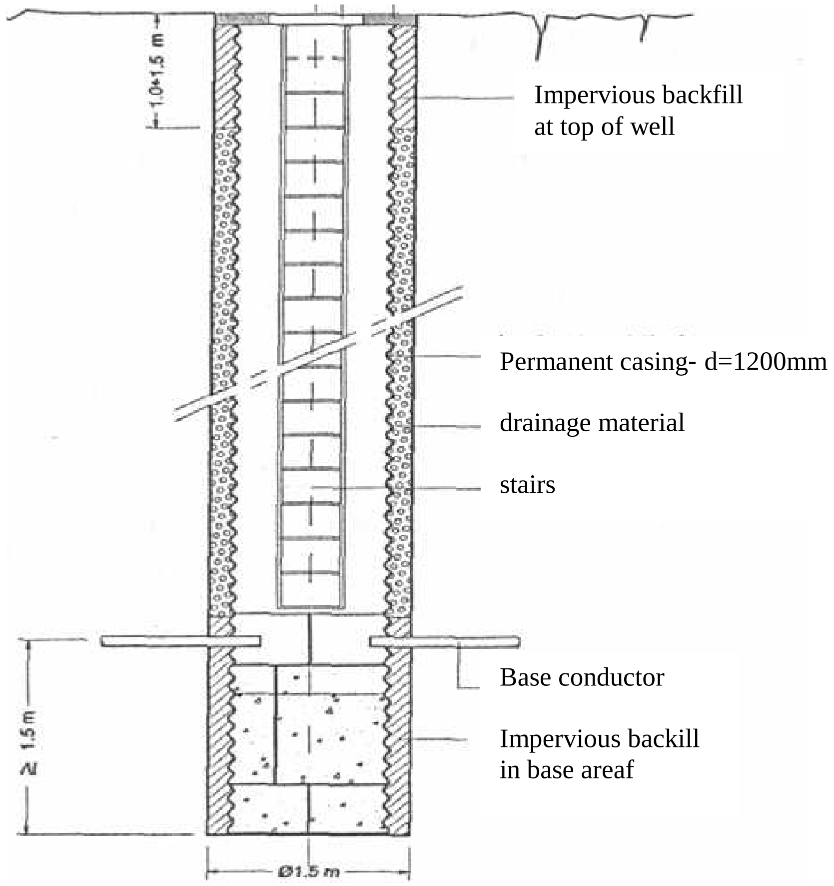
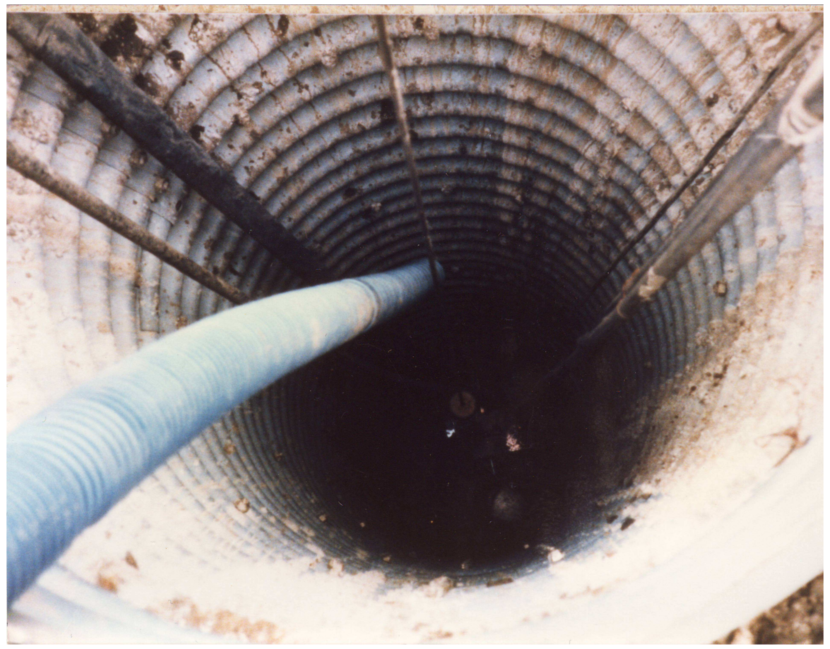


Figure 7: Well screen under construction

Figure 8: Well screen under construction (source: SGI-MI project files)

Figure 9: Drilling equipment for base conductor (source: SGI-MI project files)

Figure 10: Drilling base conductor in progress (source: SGI-MI project files)



MITIGATION THROUGH REDUCTION OF HAZARD			
4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE			
4.5.2 VERTICAL MEDIUM DIAMETER (1200-1500mm) WELLS- GRAVITY DRAINAGE THROUGH BASE CONDUCTOR			
APPLICABILITY			
Class	**Descriptor**	**Rating**	**Notes**
Type of movement (Cruden & Varnes, 1996)	Falls	0	This system usually is adopted to stabilize landslides with deep slip surface.
	Topples	0	
	Slides	6	
	Spreads	4	
	Flows	2	
Material	Earth	8	Deep well systems are effective in a range of soil from gravel to salty fine sands.
	Debris	6	
	Rock	2	
Depth of movement	Superficial (< 0.5 m)	0	This system can reach typical depths of 20 - 30 m.
	Shallow (0.5 to 3 m)	0	
	Medium (3 to 8 m)	2	
	Deep (8 to 15 m)	8	
	Very deep (> 15 m)	4	
Rate of movement (Varnes, 1978)	Moderately to fast	0	The steady-state condition is attained when the cone of depression reaches the equilibrium; this time is a function of the aquifer properties.
	Slow	2	
	Very slow	8	
	Extremely slow	8	
Groundwater	Artesian	4	This system is suitable for high freatic level.
	High	8	
	Low	6	
	Absent	0	
Surface water	Rain	2	This system is not suitable to drainage shallow water.
	Snowmelt	2	
	Localized	0	
	Stream	0	
	Torrent	0	
	River	0	
Maturity		8	Technique and design processes are well established and widely used in suitable conditions.
Reliability		7	Good performance depends strongly on the maintenance of the discharge pipe to allow gravity drainage
Implementation		6	Large spaces and good access required for construction of well at 6 – 8 m spacing
Typical Cost		4	Costs are very high, depending on number of wells along an array; also costs for maintenance of the discharge pipes at the bottom could be high

Note
Ratings are given on a scale of 1 to 10; the higher the grade, the most suitable is the specific method under consideration to use in landslides of the given characteristics, evaluated individually. Overall suitability to specific case under consideration may be obtained by a weighted average of these ratings, with user defined weights. Zero rating means "not applicable"

MITIGATION THROUGH REDUCTION OF HAZARD

4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE

4.5.2 VERTICAL MEDIUM DIAMETER (1200-1500mm) WELLS- GRAVITY DRAINAGE THROUGH BASE CONDUCTOR

References:

Barile A., Leonetti F., Silvestri F., Troncone A. TASK 2 – Progetto VIA "Riduzione della Vulnerabilità Sismica dei Sistemi Infrastrutturali ed Ambiente Fisico" Vulnerabilità dell'Ambiente Fisico: INTERVENTI DI RIDUZIONE DEL RISCHIO DI INSTABILITÀ DEI PENDII: TIPOLOGIE E METODI DI DIMENSIONAMENTO. Unical.

Beer P., Hegg U., Manassero V. (1992). "landslide stabilization at Ancona, Italy, dy deep drainage wells". In 6th International Symposium on Landslides, Christchurch, New Zealand, 10-14 February, 663-670.

Gullà G., Antropico L., Ferrari E., Sorriso-Valvo M., Tansi C., Terranova O., Aceto L., Nice-foro D., (2003). *"Linee guida per interventi di stabilizzazione di pendii in aree urbane da riqualificare", POP 1994/99 (Regione Calabria – Fondi UE), Misura 4.4 "Ricerca Scienti-fica e Tecnologica". Rubettino Editore Soveria Manelli.*

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Manassero V. (2001). "Il consolidamento dei pendii mediante drenaggio profondo". Convegno su rilevamento e tutela del territorio. Hydrogeo, Rimini 9-11 May, Maggioli Ed., 483-496.

MITIGATION THROUGH REDUCTION OF HAZARD

4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE

4.5.3 VERTICAL LARGE DIAMETER (>2000mm) WELLS – GRAVITY DRAINAGE THROUGH BASE CONDUCTOR

Description

This technique consists of installing large diameter (≥ 2000 mm, Figure 2) wells similar in every respect to the inspection wells described in fact sheet 4.5.2, except that the 1200 mm diameter casing of the standard inspection well is installed inside a separate, 1800 mm diameter, permanent corrugated hot galvanized steel casing, which is installed first, filling the annular space between the external casing and the soil with drainage material, simultaneously extracting the temporary casing used to support the hole during drilling. (Figure 3).
Once installation of the external casing is complete, it is possible to install a reinforcment cage and the inner casing, filling the annular space between the two casings with concrete. This makes these wells resistant to bending and shear, such that they double up as structural elements transferring loads from the landslide mass to more competent strata below. Accordingly, these wells are often referred to as “structural wells”.
The hydraulic connection between the external filter and the inner cavity of the well is provided by one or more short PVC pipe(s) placed in short horizontal drillholes across the two steel linings and the concrete in between. The well is then completed with the discharge pipe at the bottom, sub-horizontal drains, stairs, sealing of head and base and manhole cover.
Prior to the introduction of directional drilling, this type of well was used to install longer sub-horizontal drains than would have been possible otherwise.
Depending on the specific requirements of the project, these wells can be used in isolation as described above, as arrays of structural wells or in combination with the “hydraulic wells” described in fact sheet 4.5.2.

Design

For the structural design of these wells, where they intersect the shear plane and toe into competent material, reference may be made to fact sheets 6.2 and 6.3. For their hydraulic design when used in arrays, reference may be made as far as applicable to fact sheet 4.5.2. Where additional drainage function is provided by sub-horizontal drains, the design must define the number, elevation, orientation and length of subhorizontal drains pipes. In this case reference may be made to fact sheet 4.3 for guidance on the design of the sub-horizontal drains.

Figure 2: Classification of wells

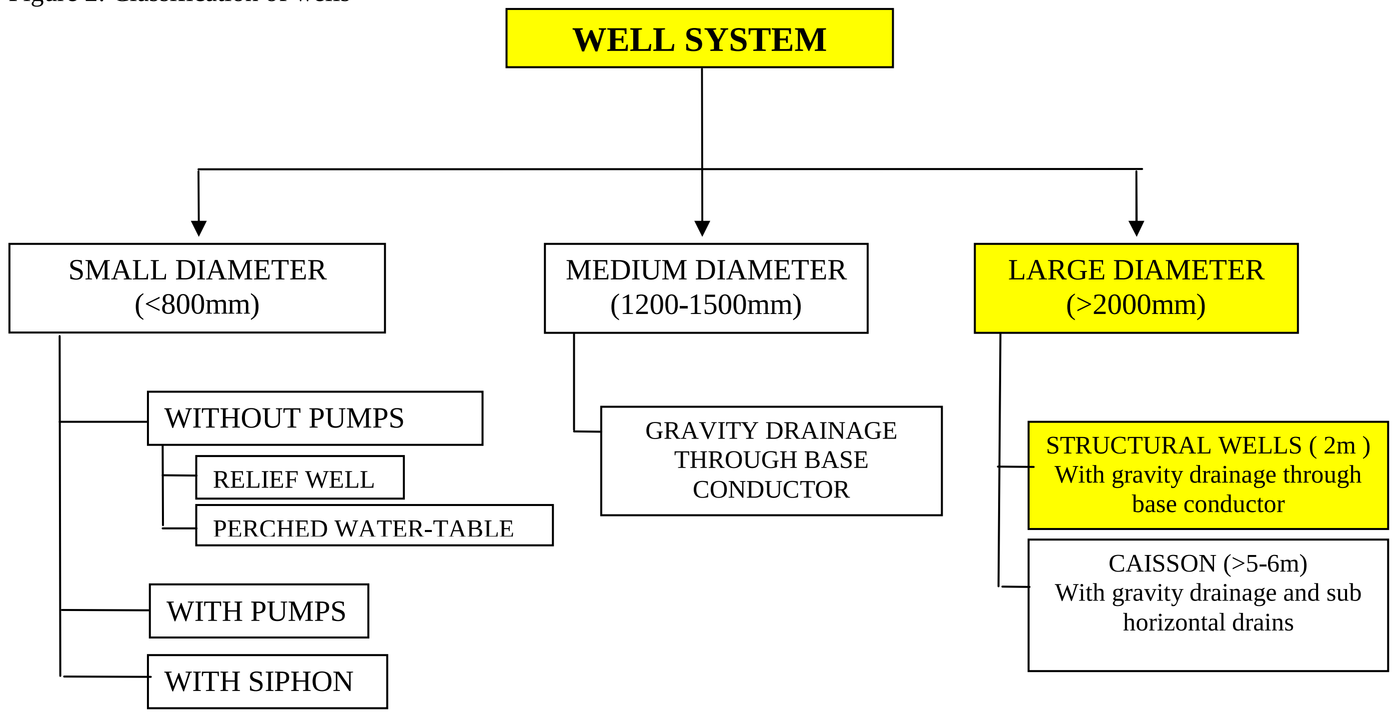


Figure 3: Beacon Hill landslides, Herne Bay,UK (Bromhead,1978). Wells deep 14 m, with a diameter of 4 m**.**

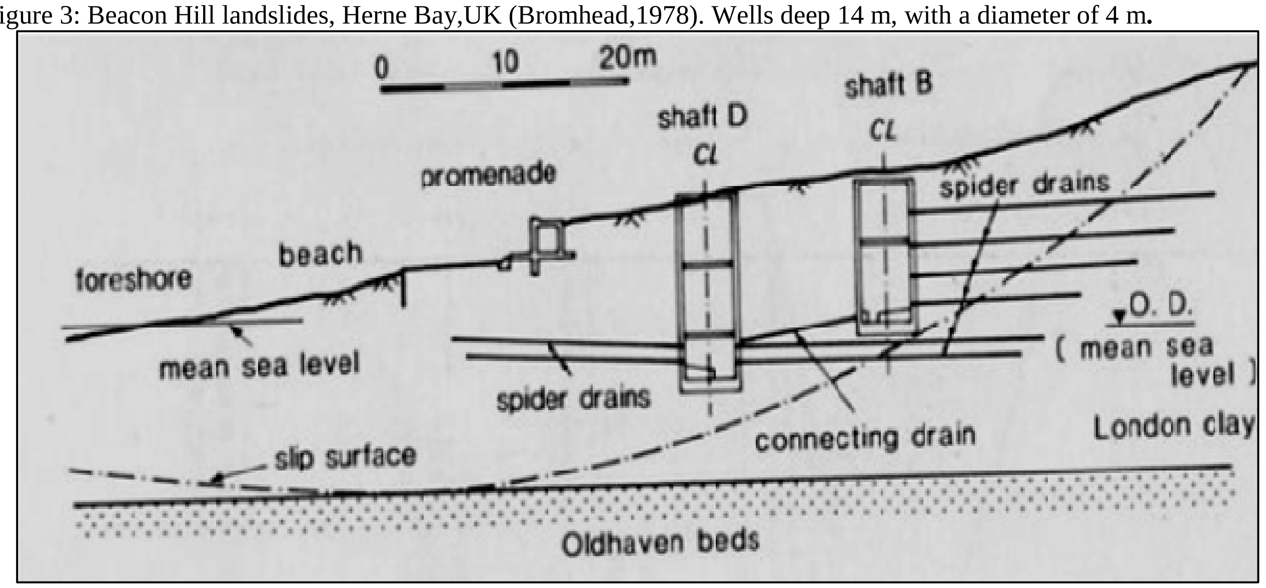
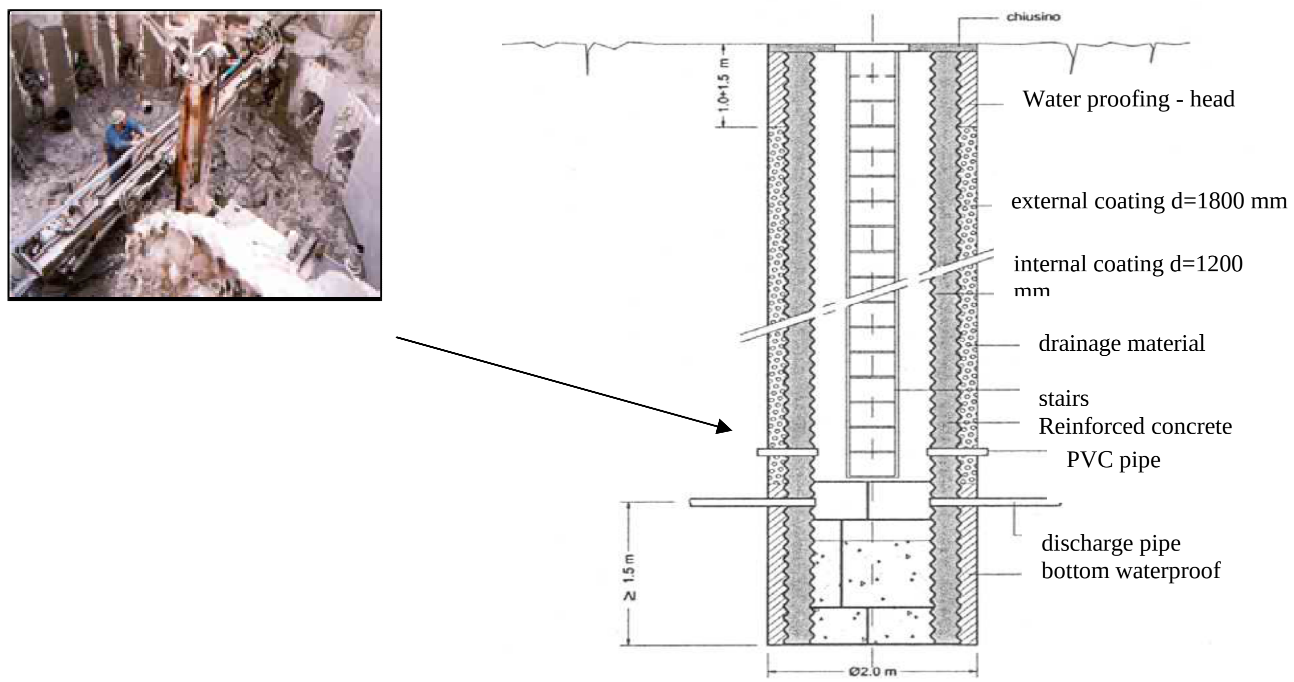


Figure 3: Typical structural well section and machinery to construct the borehole for PVC pipes (Leoni et al., 2003)



MITIGATION THROUGH REDUCTION OF HAZARD			
4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE			
4.5.3 VERTICAL LARGE DIAMETER (>2000mm) WELLS – GRAVITY DRAINAGE THROUGH BASE CONDUCTOR			
APPLICABILITY			
Class	**Descriptor**	**Rating**	**Notes**
Type of movement (Cruden & Varnes, 1996)	Falls	0	This system is usually adopted to stabilize landslides with deep slip surface.
	Topples	0	
	Slides	6	
	Spreads	4	
	Flows	2	
Material	Earth	8	Deep well systems are effective in a range of soil from gravel to salty fine sands.
	Debris	6	
	Rock	2	
Depth of movement	Superficial (< 0.5 m)	0	This system can reach typical depths of 10 - 15 m.
	Shallow (0.5 to 3 m)	0	
	Medium (3 to 8 m)	2	
	Deep (8 to 15 m)	8	
	Very deep (> 15 m)	4	
Rate of movement (Varnes, 1978)	Moderately to fast	0	The steady-state condition is attained when the cone of depression reaches the equilibrium; this time is a function of the aquifer properties.
	Slow	2	
	Very slow	8	
	Extremely slow	8	
Groundwater	Artesian	4	This system is suitable for high freatic level.
	High	8	
	Low	6	
	Absent	0	
Surface water	Rain	2	This system is not suitable to drainage shallow water.
	Snowmelt	2	
	Localized	0	
	Stream	0	
	Torrent	0	
	River	0	
Maturity		8	Technique and design processes are well established and widely used in suitable conditions.
Reliability		7	Good performance depends strongly on the maintenance of the discharge pipe and sub horizontal drains
Implementation		6	Large spaces need for wells spaced 6 m
Typical Cost		4	Costs are very high, due to the number of wells along an array; also costs for maintenance of the discharge pipes at the bottom and of the subhorizontal drains could be high

Note
Ratings are given on a scale of 1 to 10; the higher the grade, the most suitable is the specific method under consideration to use in landslides of the given characteristics, evaluated individually. Overall suitability to specific case under consideration may be obtained by a weighted average of these ratings, with user defined weights. Zero rating means "not applicable"

MITIGATION THROUGH REDUCTION OF HAZARD

4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE

4.5.3 VERTICAL LARGE DIAMETER (>1500mm) WELLS- GRAVITY DRAINAGE THROUGH BASE CONDUCTOR

References:

Barile A., Leonetti F., Silvestri F., Troncone A. TASK 2 – Progetto VIA "Riduzione della Vulnerabilità Sismica dei Sistemi Infrastrutturali ed Ambiente Fisico" Vulnerabilità dell'Ambiente Fisico: INTERVENTI DI RIDUZIONE DEL RISCHIO DI INSTABILITÀ DEI PENDII: TIPOLOGIE E METODI DI DIMENSIONAMENTO. Unical.

Bromhead EN (1978) Large landslides in London Clay at Herne Bay, Kent. Q J Eng Geol 11:291–304

Gullà G., Antropico L., Ferrari E., Sorriso-Valvo M., Tansi C., Terranova O., Aceto L., Nice-foro D., (2003). *"Linee guida per interventi di stabilizzazione di pendii in aree urbane da riqualificare", POP 1994/99 (Regione Calabria – Fondi UE), Misura 4.4 "Ricerca Scienti-fica e Tecnologica". Rubettino Editore Soveria Manelli.*

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Ziccarelli M.(2010). Stabilizzazione dei pendii mediante drenaggi. Rende, 26 Maggio 2010.

MITIGATION THROUGH REDUCTION OF HAZARD

4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE

4.5.4 CAISSON (> 5-6 m), WITH GRAVITY DRAINAGE (AND SECONDARY SUBHORIZONTAL DRAINS)

Description

Large diameter caissons (figure 1), excavated as described in fact-sheet 6.4 may be left with an open shaft and equipped with arrays of sub-horizontal microdrains, as described in fact sheets 4.3 and 4.4, to supplement their structural role with drainage. Typical vertical and horizontal sections are shown in Figures 2 and 3. Figures 4 to 7 illustrate significant details. A typical application is shown in figures 8 and 9. In theory, such caissons could be constructed ourely for their drainage function, but this is unlikely to be appropriate and economic in practice.

Additional drainage may occur along the shaft wall, if this consists of discrete columnar elements (piles, miicropiles) with a gap between them and vertical draining mats are installed adhering with the ground between adjacent piles around the perimeter of the shaft (see Figure X).

The minimum diameter of the caisson is dictated by the space required for the installation of the microdrains. Indicatively, the minimum diameter is 5 m for microdraind 20 to 30 m long and 8 to 10 m for microdrains 50 to 60 m long.

The water intercepted may be discharged connecting the wells at the base by one or two small diameter collectors, allowing the water to flow away at the base of the slope to be stabilized.Wells with diameter of 8 to 10 m, equipped with a large number of drains need large diameter discharge collectors. In this case, collectors up to 1000 mm diameter are carried out using microtunnelling technology.This type of shaft may also be used as the starting or arrival point of drainage tunnels or as otfall for deep drainage trenches.

Design

For the structural design of these caissons, reference may be made to fact sheets 6.4. Where the main drainage function is provided by sub-horizontal drains, the design must define the number, elevation, orientation and length of subhorizontal drains pipes. In this case reference may be made to fact sheet 4.3 for guidance on the design of the sub-horizontal drains.

The minimum section of the base conductor are determined by conventional hydraulic calculations based on the required drawdown and the associated flow. Spare capacity should be provided, to minimize maintenance requirements

Figure 1: Classification of wells

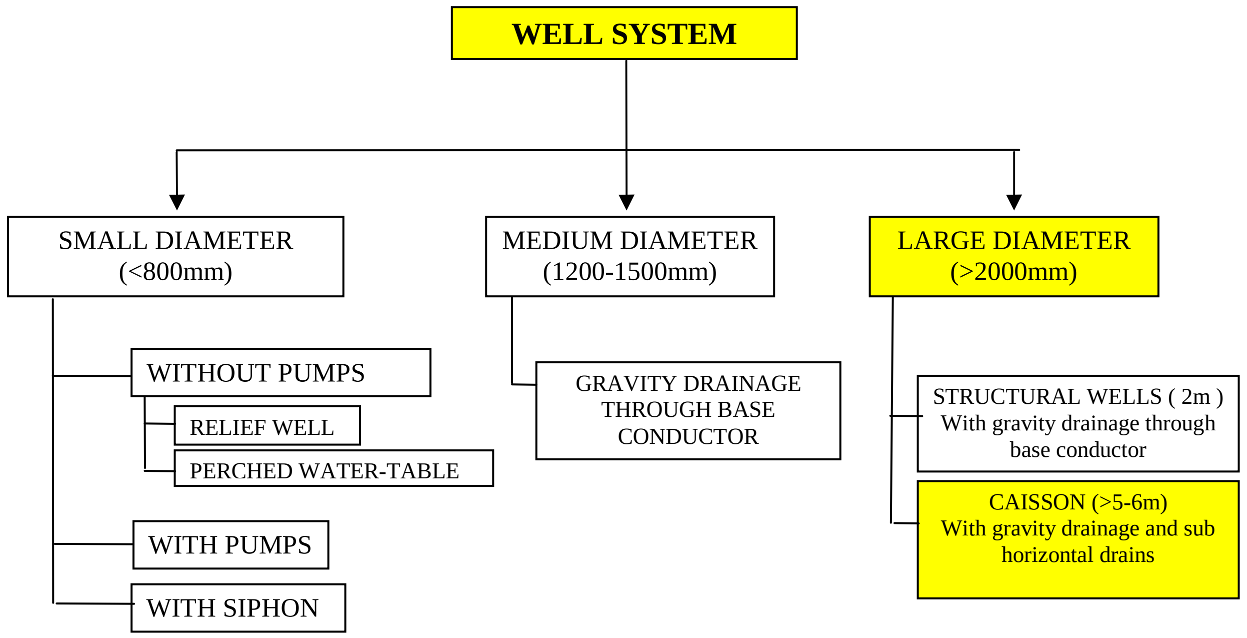


Figure 2: Typical large diameter caisson with drainage function: vertical Section (source: SGI-MI project files)

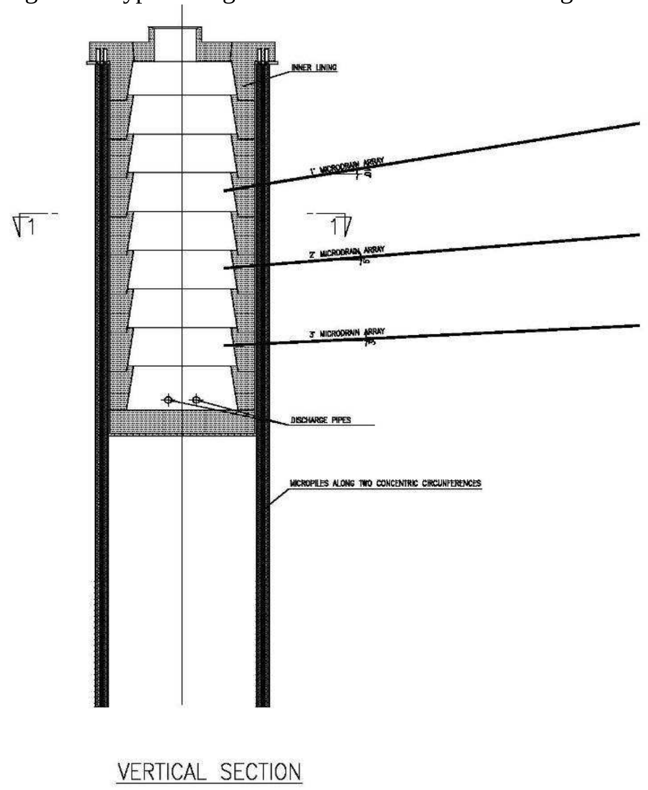
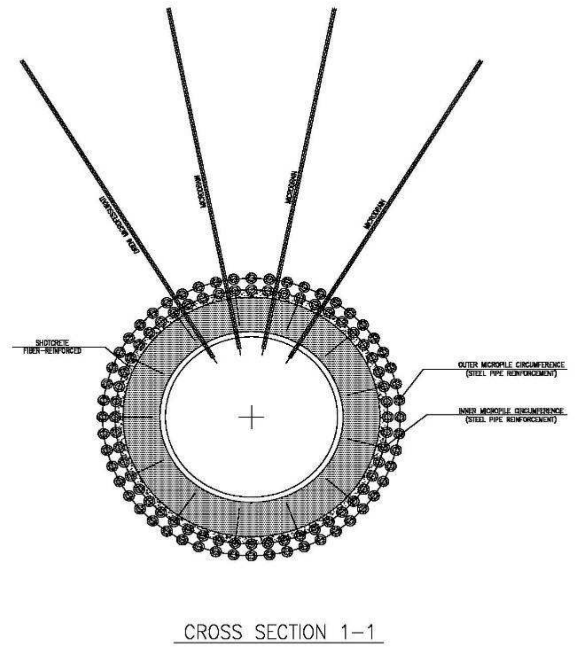


Figure 3: Typical large diameter caisson with drainage function: Section 1-1 on horizontal plane (source: SGI-MI project files)



MITIGATION THROUGH REDUCTION OF HAZARD

4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE

4.5.4 CAISSON (> 5-6 m), WITH GRAVITY DRAINAGE (AND SECONDARY SUBHORIZONTAL DRAINS)

Figure 4: Well excavation; support structure consists of micropiles and steel ribs (source (SGI-MI project file

Figure 5: detail of microdrain heads (source (SGI-MI project file

Figure 6: Well with secondary drainage; structure consists of discrete piles and concrete ribs (source (SGI-MI project file

Figure 7: Vertical drain mats in adherence with the ground between piles (source (SGI-MI project file

MITIGATION THROUGH REDUCTION OF HAZARD

4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE

4.5.4 CAISSON (> 5-6 m), WITH GRAVITY DRAINAGE (AND SECONDARY SUBHORIZONTAL DRAINS)

Figure 8: Typical application of caissons with combined structural and drainage function, provided by arrays of sub-horizontal microdrains and gravity discharge from base of hollow shaft. Plan and Longitudinal Section along basal discharge conduit (source: SGI-MI project files)

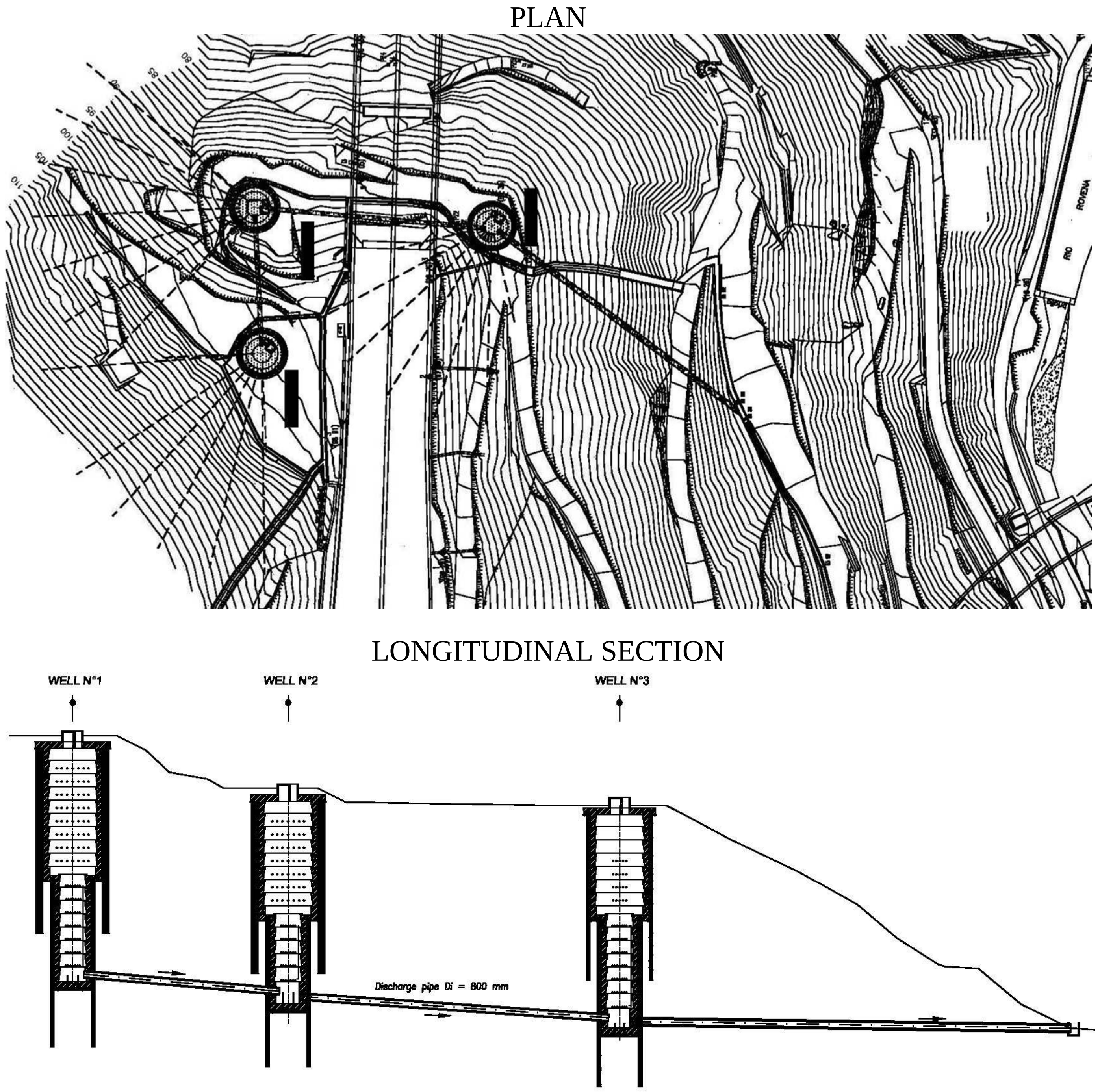
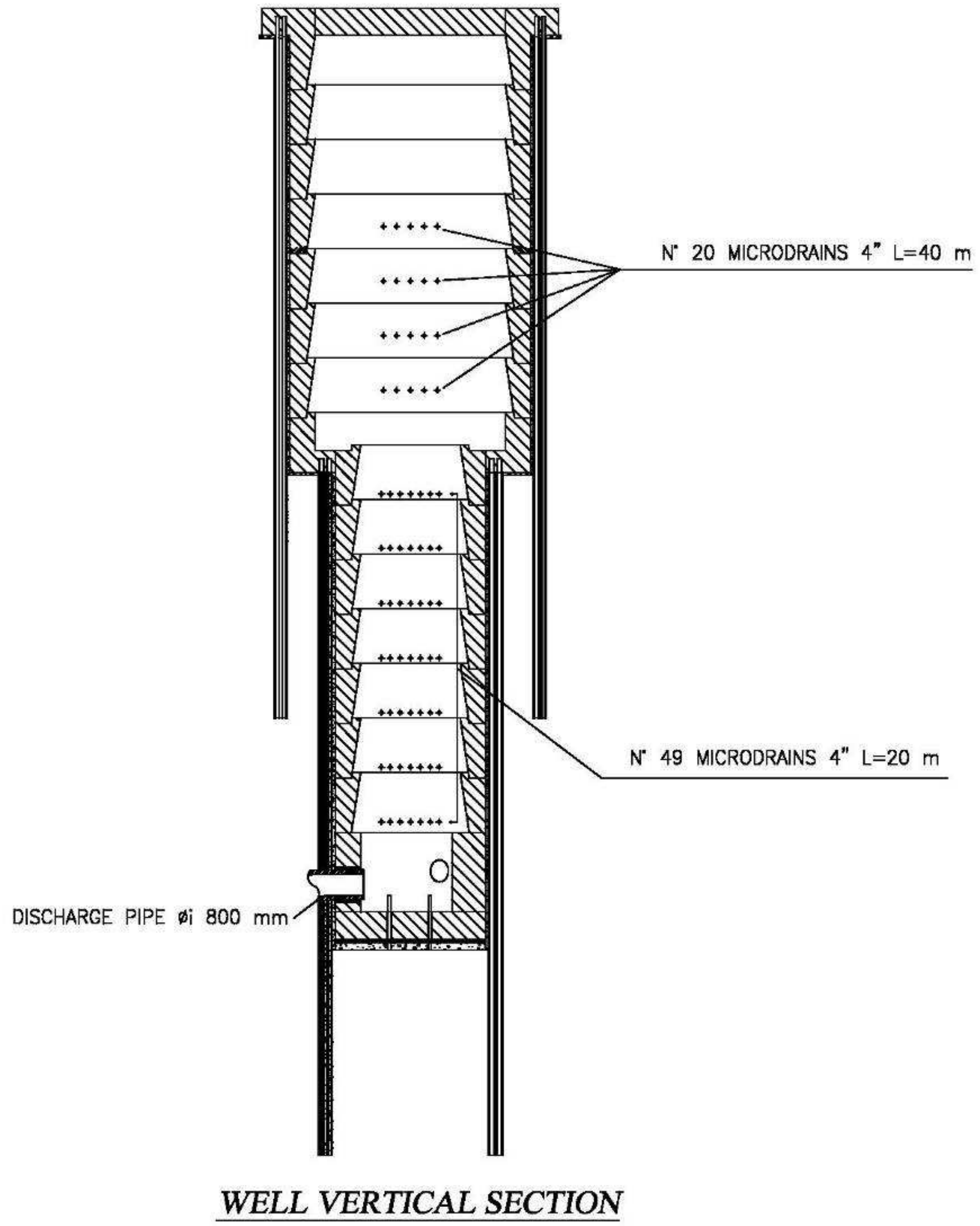


Figure 9: Telescopic caisson with combined structural and drainage function, provided by arrays of sub-horizontal microdrains and gravity discharge from base of hollow shaft (source: SGI-MI project files)



MITIGATION THROUGH REDUCTION OF HAZARD

4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE

4.5.4 CAISSON (> 5-6 m), WITH GRAVITY DRAINAGE (AND SECONDARY SUBHORIZONTAL DRAINS)

APPLICABILITY

Class	Descriptor	Rating	Notes
Type of movement (Cruden & Varnes, 1996)	Falls	0	This system is usually adopted to stabilize landslides with deep slip surface.
	Topples	0	
	Slides	6	
	Spreads	6	
	Flows	4	
Material	Earth	8	Deep well systems are effective in a range of soil from gravel to silty fine sands.
	Debris	6	
	Rock	2	
Depth of movement	Superficial (< 0.5 m)	0	This system can reach typical depths of 10 - 15 m.
	Shallow (0.5 to 3 m)	0	
	Medium (3 to 8 m)	0	
	Deep (8 to 15 m)	6	
	Very deep (> 15 m)	8	
Rate of movement (Varnes, 1978)	Moderately to fast	0	The steady-state condition is attained when the cone of depression reaches the equilibrium; this time is a function of the aquifer properties.
	Slow	2	
	Very slow	8	
	Extremely slow	8	
Groundwater	Artesian	4	This system is suitable for high freatic level.
	High	8	
	Low	6	
	Absent	0	
Surface water	Rain	2	This system is not suitable to drainage shallow water.
	Snowmelt	2	
	Localized	0	
	Stream	0	
	Torrent	0	
	River	0	
Maturity		7	Technique and design processes are well established and widely used in suitable conditions.
Reliability		7	Good performance depends strongly on the maintenance of the discharge pipe and sub horizontal drains
Implementation		7	Large spaces need for shafts 6 to 10 m in diameter, plus additional working space.
Typical Cost		2	The range of costs is very large and depends on many factors as the well dimensions, the soil nature, the number and the length of drains, etc.

Note
Ratings are given on a scale of 1 to 10; the higher the grade, the most suitable is the specific method under consideration to use in landslides of the given characteristics, evaluated individually. Overall suitability to specific case under consideration may be obtained by a weighted average of these ratings, with user defined weights. Zero rating means "not applicable"

MITIGATION THROUGH REDUCTION OF HAZARD	
4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE	
4.5.4 CAISSON (> 5-6 m), WITH GRAVITY DRAINAGE (AND SECONDARY SUBHORIZONTAL DRAINS)	
References:	

MITIGATION THROUGH REDUCTION OF HAZARD

4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE

4.6 DRAINAGE TUNNELS, ADITS, GALLERIES, WITH SECONDARY DRAINS OR AS OUTLET FOR WELLS

Description

Drainage galleries constitute a rather expensive stabilization measure for large, deep landslide movements (30-35 m), to be used where the subsoil is unsuitable for trenches or drainage wells and when it is impossible to work on the surface owing to lack of space for the work machinery. In fact galleries are expensive, if compared with inclined bored drains but they may be advantageous where seepage takes place from closely-spaced fissures or laminations in a rock formation. Moreover, these drainage systems have to be built on the stable part of the slope (Figs. 1-2).

The gallery can be tunnelled to intercept the source of seepage and then continued along the water-bearing horizon to the extent necessary to achieve the lowering of piezometric pressures behind the slope. Drainage galleries provide a means of access for supplementary stabilization measures such as transverse adits, inclined bored drains, or grouting. The drainage systems are placed inside the galleries and are made up of micro drains, with lengths that can reach 50–60 m and are spatially oriented in suitable directions. The sizes of galleries are conditioned by the need to insert the drain drilling equipment. For this reason the minimum transversal internal size of galleries vary from a minimum of 2 m, when using special reduced size equipment, to at least 3.5 m, when using traditional equipment (Fig. 3).

Galleries are constructed on an upward gradient to permit drainage by gravity towards the portal through a piped drain constructed beneath the floor of the gallery. The drain should have a removable cover for easy inspection and maintenance.

Where a gallery is constructed in highly-weathered rocks, permanent support is required in the form of reinforced concrete liling. In this case, the permanent lining should be surrounded with a properly designed drainage filter so that there is a good hydraulic connection with the material being drained. Weepholes then have to be provided through the lining in order to drain the filter.

Design

Drainage galleries can be very effective in dewatering the slope, because of high surface area exposed for drainage. They are however expensive; therefore careful consideration about costs and benefits is required. The position and size of the gallery is important as shown by Sharp (1970). In fact the knowledge of the ground-water flow is necessary to design the position, the path, and the size of the galleries and the length of any micro drains installed inside galleries.

The flow to drain apart from the permeability of surrounding soils depends on the size and the inner surface of galleries.

The flow per unit length of the gallery, with only one side at the contact with freatic aquifer can be calculated as (with f: = hydraulic permeability, Milano, 2005):

$$q = \frac{f}{2L}\delta(2H - \delta)$$

.

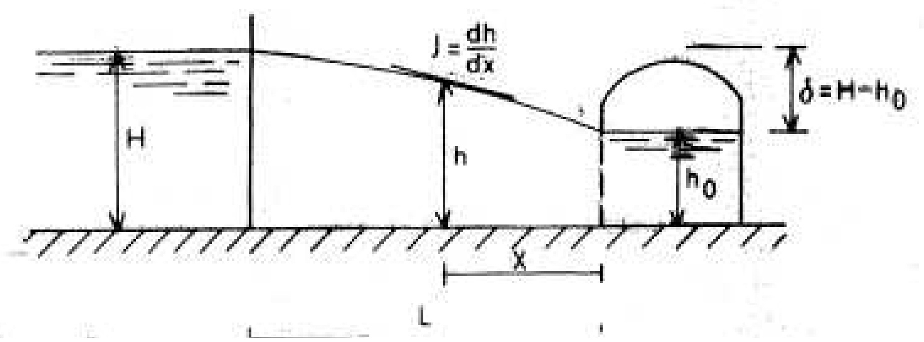


Figure 1: Drainage galleries with drain pipes

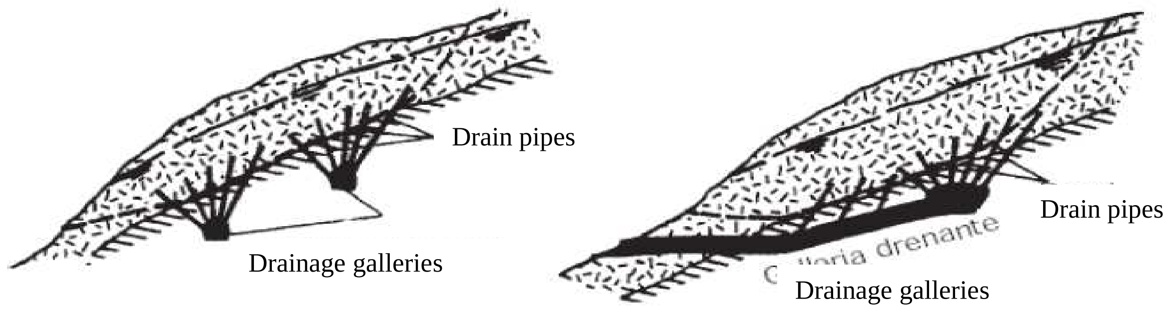
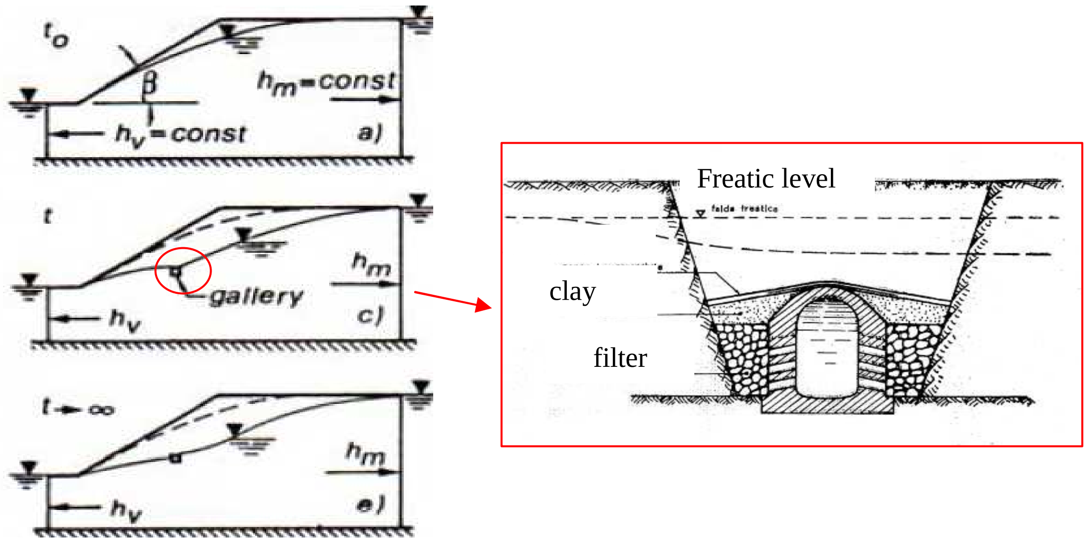


Figure 2: a) Typical shape of freatic level before and after the construction of galleries; b) typical transversal section of galleries



a) b)

Figure 3: Microtunneling system to insert microdrains.

MITIGATION THROUGH REDUCTION OF HAZARD			
4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE			
4.5.3 DRAINAGE TUNNELS, ADITS, GALLERIES, WITH SECONDARY DRAINS OR AS OUTLET FOR WELLS			
APPLICABILITY			
Class	**Descriptor**	**Rating**	**Notes**
Type of movement (Cruden & Varnes, 1996)	Falls	2	Drainage galleries constitute an effective but expensive mitigation measure for landslide movements.
	Topples	4	
	Slides	6	
	Spreads	6	
	Flows	6	
Material	Earth	6	They may be advantageous where seepage takes place from closely-spaced fissures or laminations in a rock formation.
	Debris	6	
	Rock	6	
Depth of movement	Superficial (< 0.5 m)	0	rainage galleries constitute a mitigation measure for large, deep landslide movements (30-35 m).
	Shallow (0.5 to 3 m)	0	
	Medium (3 to 8 m)	2	
	Deep (8 to 15 m)	6	
	Very deep (> 15 m)	8	
Rate of movement (Varnes, 1978)	Moderately to fast	4	The steady-state condition is attained when the cone of depression reaches the equilibrium; time is a function of the acquifer properties.
	Slow	8	
	Very slow	8	
	Extremely slow	8	
Groundwater	Artesian	6	This system is very suitable for freatic acquifer.
	High	8	
	Low	8	
	Absent	0	
Surface water	Rain	0	This system is not suitable to drainage the shallow water.
	Snowmelt	0	
	Localized	0	
	Stream	0	
	Torrent	0	
	River	0	
Maturity		7	Technique and design processes are well established and widely used in suitable conditions.
Reliability		7	They may be advantageous where seepage takes place from closely-spaced fissures or laminations in a rock formation.
Implementation		6	The same technologies used for tunnels are suitable. Recent applications are carried out by microtunneling system (Angeli & Pontoni, 2002).
Typical Cost		1	They are the most expensive mitigation system for slope stability and are built where the slope is unsuitable for trenches or drainage wells and when it is impossible to work on the surface owing to a lack of space for the work machinery.

Note
Ratings are given on a scale of 1 to 10; the higher the grade, the most suitable is the specific method under consideration to use in landslides of the given characteristics, evaluated individually. Overall suitability to specific case under consideration may be obtained by a weighted average of these ratings, with user defined weights. Zero rating means "not applicable"

MITIGATION THROUGH REDUCTION OF HAZARD

4 MODIFYING THE GROUNDWATER REGIME - DEEP DRAINAGE

4.6 DRAINAGE TUNNELS, ADITS, GALLERIES, WITH SECONDARY DRAINS OR AS OUTLET FOR WELLS

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MITIGATION THROUGH REDUCTION OF HAZARD

FACT SHEET 5

MODIFYING THE MECHANICAL CHARACTERISTICS OF THE UNSTABLE MASS

MITIGATION THROUGH REDUCTION OF HAZARD

5 MODIFYING THE MECHANICAL CHARACTERISTICS OF UNSTABLE MASS

5.0 GENERAL

Description

Many methods can be used to modify the mechanical characteristics of unstable masses. The majority of these methods have been developed and are widely used for improving soil in civil engineering works, such as building foundations, bridge foundations and embankments, located in areas of low to negligible inclination. The applicability and relevance of these methods to slope stabilization depends on the required depth of treatment, the soil type to be trated and the equipment necessary to carry out the treatment. While a large number of methods exist, only those specifically applicable to landslide stabilization are described here and in the attached fact sheets..

Typically, shallow unstable or weak soil may be stabilized by use of vegetation, surface substitution or surface compaction.

Most of the methods providing increased stability at greater depths are based on forming inclusions (typically, columns of some sort) with higher strength and/or stiffness than the surrounding soil, either modifying the soil itself, or in such a way that also the parent soil between the inclusions is modified.

Deep compaction methods achieve this by increased soil density (vibrocompaction) or by introducing coarser materials (stone colums: vibroreplacement and/or vibrodisplacement).

Deep soil mixing, permeation grouting and jet-grouting increase the strength of the soil using admixtures to fill the voids and to bind of soil grains, the three methods differing mainly in the technique used to achieve the required penetration of the admixture in the soil matrix. The advantages of mixing and grouting methods is that the soil treatment may be limited to the area of failure, limiting the use of stabilizing materials and costs.

Grids of isolated, tangential or secant inclusions are used, depending on the type of treatment and degree of improvement required. Isolated unreinforced inclusions are susceptible to failing in bending and in tension (Figure 1). Where the method of improvement allows it is therefore always advisable to use tangential or secant inclusions to form panels aligned with the longitudinal axis in the direction of movement (Figure 2).

Table 1 summarizes the soil conditions in which the different methods of ground improvement applicable in landslide stabilization are typically effective and applied in practice. Further details are provided in the relevant fact sheets.

Table 1: Indicative range of applicability of different methods of ground improvement to landslide stabilization.

Method	Grain size distribution/characteristic where typically the method is effective
Vibrocompaction	From gravels to slightly silty sands
Stone coulms: Vibroreplacement and vibrodisplacement	Silty sands to sandy silts (#)
Mechanical soil mixing	From sands to clays
Permeation grouting	Coarse to medioun sands, fractured rock (*)
Jet grouting	From sands to clays
Modification of groundwater chemistry (e.g. lime piles)	Clays

Notes:
(#) in finer soils these techniques may be used for other purposes, but typically they would not be the method of choice for landslide stabilization
(*) ultra fine, low viscosity or chemical binders are required for soil of low permeability such as fine sands and silty sands

For completeness, two additional methods are mentioned here for which no fact sheet is provided since they are seldom used: elctroosmosis and thermal treatment.

Electroosmosis

Electroosmosis consists in forcing drainage in fine grained soils by the application of an electric potential. Water is dragged by the cations towards the cathode, where it is extracted (Mitchell, 1976). The efficiency an the economics of electroosmosis depend on the water transported per unit charge passed (cubic metres per hour per Ampere). The rate of water movement depends on the applied electric field, the flow resistance of the soil and the frictional drag of the ions on the water molecules.

Figure 1) Failure in bending of isolated inclusions subject to inclined loading (source: Terashi, 2003)

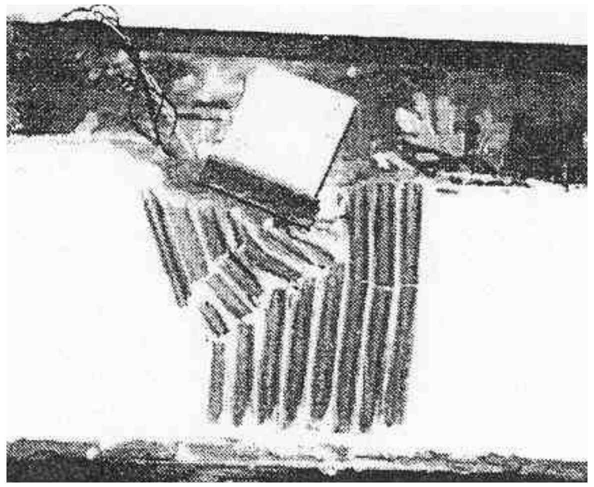
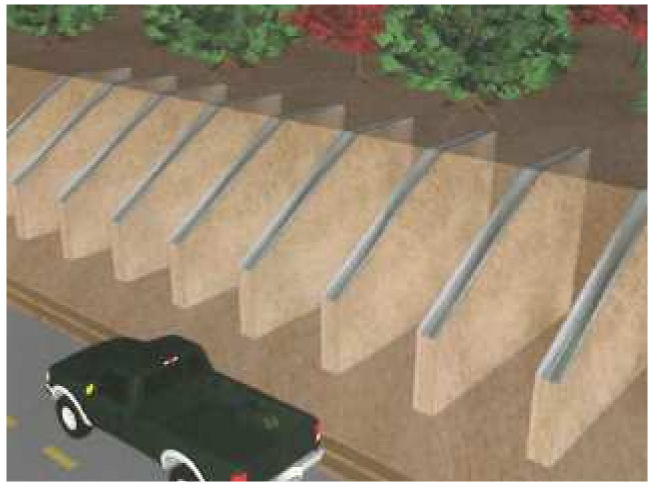


Figure 2) Panels formed by tangential or secant inclusions to resist inclined loading (source: Soletanche Bachy)



The greater the difference between the concentrations of cations and anions, the greater the net drag on the water towards the cathode.The classic application of electroosmosis has been to stabilize landslides and slopes in fine grained soils. The classic papers on electroosmosis were published by Casagrande (1948, 1952, 1953). An early case case history was reported by Casagrande et al. (1961) in which electroosmosis was adopted to stabilize a 30 m high slope in organic silt along the Trans Canada Highway in Ontario. Casagrande et al. (1981) described the use of electroosmosis to stabilize a slope for a 80 m deep exacavation for the cut-off trench of a dam in British Columbia.

Despite some success, this technique has not received widespread usage, probably because of the high installation and operation costs and of some remaining technical uncertainties about the process.

Lo et al. (1991a, 1991b) used specifically designed copper electrods to prevent gas accumulation around the anode and to allow free water to flow from the cathode without pumping; with a significant reduction in both installation and electricity costs compared to previous electroosmosis installations.

Thermal treatment

Thermal treatment can involve either heating or freezing the soil to modify its characteristics.

Heat traeatment was used in Rumania to stabilize clay slopes (Beles, 1957). Experiments with thermal treatment in clays and loess soils were carried out in the former Soviet Union (Gedney and Weber, 1978).

High temperatures dry out the soil and tend to fuse fine grained particles, leading to a permanent increases in shear strength of the soil and a consequent increases in slope stability. The high cost of this technique has precluded its use on all but the most experimental slope remediation problems.

Ground freezing has developed in recent years to be a very effective technique for temporary stabilization of large excavations and tunnels; one of the most complete treatises on ground freezing is by Jessberger (1979). While the frozen mass is relatively stable, significant disturbance, deformation and stress changes can occur during freezing and thawing, especially in fine grained soils; accordingly, ground freezing must be applied with great caution.

Ground freezing may be a useful technique to overcome temporary construction problems, such as the difficulties encountered in excavating drainage galleries at a landslide on the Danube in Novi Sad, Serbia (Vasic, 2007; Djogo and Vasic, 2011), but it is unlikely to be used as a mitigation measure in itself

Design

Specific consideration on design for each method are included in the relevant fact sheets

MITIGATION THROUGH REDUCTION OF HAZARD

5 MODIFYING THE MECHANICAL CHARACTERISTICS OF UNSTABLE MASS

5.0 GENERAL

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MITIGATION THROUGH REDUCTION OF HAZARD

5 MODIFYING THE MECHANICAL CHARACTERISTICS OF UNSTABLE MASS

5.1 VEGETATION – MECHANICAL EFFECTS

Description

Trees have been planted on many slopes worldwide specifically to increase slope stability. For example, 60.000 fast-growing acacia and gmelina seedlings were planted in an effort to stabilize the historic Cucaracha landslide in the Gaillard Cut of the Panama Canal when it was reactivated in 1986, almost blocking the canal (Rivera, 1991; Berman, 1991).

Vegetation is widely believed to improve the stability of slopes, especially on steep slopes and with respect to superficial or shallow movements. However, it can take a long time to become effective at depth and it can also have negative effects, as summarized in Table 1 (Greenway, 1987; Wu, 1995).

Table 1: Mechanical and hydrogeological effects of vegetation on slope stability (Wu, 1995)

Process	Type	Effect on stability
1. Roots increase permeability, increase infiltration, and thereby increase pore pressure	Hydrological	Negative
2. Vegetation increases interception and evapotranspiration, and thereby reduce pore pressure	Hydrological	Positive
3. Vegetation increases weight or surcharge, and thereby increases load on slope	Mechanical	Negative
4. Vegetation increases wind resistance, and thereby increases load on slope	Mechanical	Negative
5. Roots reinforce soil, and thereby increase strength	Mechanical	Positive

Information on the mechanical and hydrological effects of vegetation is provided by the Hong Kong Geotechnical Manual for Slopes (Geotechnical Control Office, 1984), reflecting one of the most comprehensive research programs in the world on the engineering role of vegetation for slope stabilization (Barker, 1991).

The net positive contribution of vegetation to slope stability is supported by a number of case studies where slope failures could be attributed to the loss of reinforcement provided by the tree roots (Wu et al., 1979; Riestenberg and Sovonick-Dunford, 1983; Riestenberg, 1987), while Greenwood et al. (2004) reported a 10% increases in the Factor of Safety of vegetated slopes compared to non-vegetated slopes.

From a mechanical point of view, vegetation can improve the stability of slopes through the anchoring or reinforcement effect provided by the roots Wu (1995) – Figure 1. The governing factors are the mechanical properties (tensile strength and elastic modulus) of the roots and their density in the shear zone. The anchoring effect of roots depends on the type of vegetation; the roots have different properties and grow differently from plant to plant (Figure 2); a denser network of roots in the soil will fovour stability and for a given species the diameter of the roots will determine the amount of stress a root can take before breaking.

Notable studies on the reinforcement effects of roots on vegetated slope have been conducted by Greenway et al. (1984), Greenway (1987) and Yin et al. (1988). Wu (1995) shows that roots left after logging continue to have a positive effect on slope stability for many years, with their tensile strength reducing gradually, but it takes time for new trees to establish a new stabilizing root system.

Greenwood et al. (2007) highlight that vegetation may also result in increased suction (negative pore pressure) in unsaturated soil, potentially increasing the apparent cohesion of the soil.

Reference shall be made to the fact sheets in section 1 of this Annex for further detailed description of applicable techniques and discussion of the basis of design for the use of vegetation to improve slope stability.

For considerations on the hydrological effects of vegetation on soil stability reference may be made to fact sheet 3.5 of this Annex.

Figure 1: Stabilization of a slope by the root system of a banyan tree, Hawaï (source: Rogers., 1992)

Figure 2: Selected species can develop significant root systems (source www.pratiarmati.it)

MITIGATION THROUGH REDUCTION OF HAZARD			
5 MODIFYING THE MECHANICAL CHARACTERISTICS OF UNSTABLE MASS			
5.1 VEGETATION – MECHANICAL EFFECTS			
APPLICABILITY			
Class	**Descriptor**	**Rating**	**Notes**
Type of movement (Cruden & Varnes, 1996)	Falls	0	Both rotational and translational.
	Topples	0	
	Slides	4	
	Spreads	0	
	Flows	0	
Material	Earth	8	Careful selection of species is required for applications on rock, where roots may actually open fractures favouring water ingress and instability
	Debris	4	
	Rock	2	
Depth of movement	Superficial (< 0.5 m)	8	Limited by root penetration
	Shallow (0.5 to 3 m)	4	
	Medium (3 to 8 m)	0	
	Deep (8 to 15 m)	0	
	Very deep (> 15 m)	0	
Rate of movement (Varnes, 1978)	Moderately to fast	2	Seeding can be appplied remotely, by helicopter if necessary. However, it needs time to become established and this may limit application in moderately to fast movements
	Slow	6	
	Very slow	8	
	Extremely slow	8	
Groundwater	Artesian	8	Species must be selected to suit agronomical conditions. Irrigation may be neccessary in arid soils.
	High	8	
	Low	4	
	Absent	2	
Surface water	Rain	8	May be used to stabilize banks of slow watercourses, but it requires special techniques.
	Snowmelt	8	
	Localized	6	
	Stream	4	
	Torrent	0	
	River	4	
Maturity		6	Impact on mechanical aspects of slope processes not yet fully established. Strong reliance on empiricism.
Reliability		4	Needs significant maintenance, especially in early stages; inappropriate species selection could be ineffective or even detrimental to stability
Implementation		8	Application on steep slopes or moderately to fast slides may be done remotely
Typical Cost		8	Relatively low installation costs, but it may require significant maintenance or even irrigation

Note
Ratings are given on a scale of 1 to 10; the higher the grade, the most suitable is the specific method under consideration to use in landslides of the given characteristics, evaluated individually. Overall suitability to specific case under consideration may be obtained by a weighted average of these ratings, with user defined weights. Zero rating means ”not applicable”

MITIGATION THROUGH REDUCTION OF HAZARD

5 MODIFYING THE MECHANICAL CHARACTERISTICS OF UNSTABLE MASS

5.1 VEGETATION – MECHANICAL EFFECTS

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MITIGATION THROUGH REDUCTION OF HAZARD

5 MODIFYING THE MECHANICAL CHARACTERISTICS OF UNSTABLE MASS

5.2 SUBSTITUTION

Description
The stability of slopes may be increased by substitution of the original materials with materials of higher strength and, possibly, higher permeability. In the latter case, provided that drainage is permitted, additional increases in shear resistances may derive from changes of effective stresses due to the lowering of pore pressures. It is possible to distinguish between:

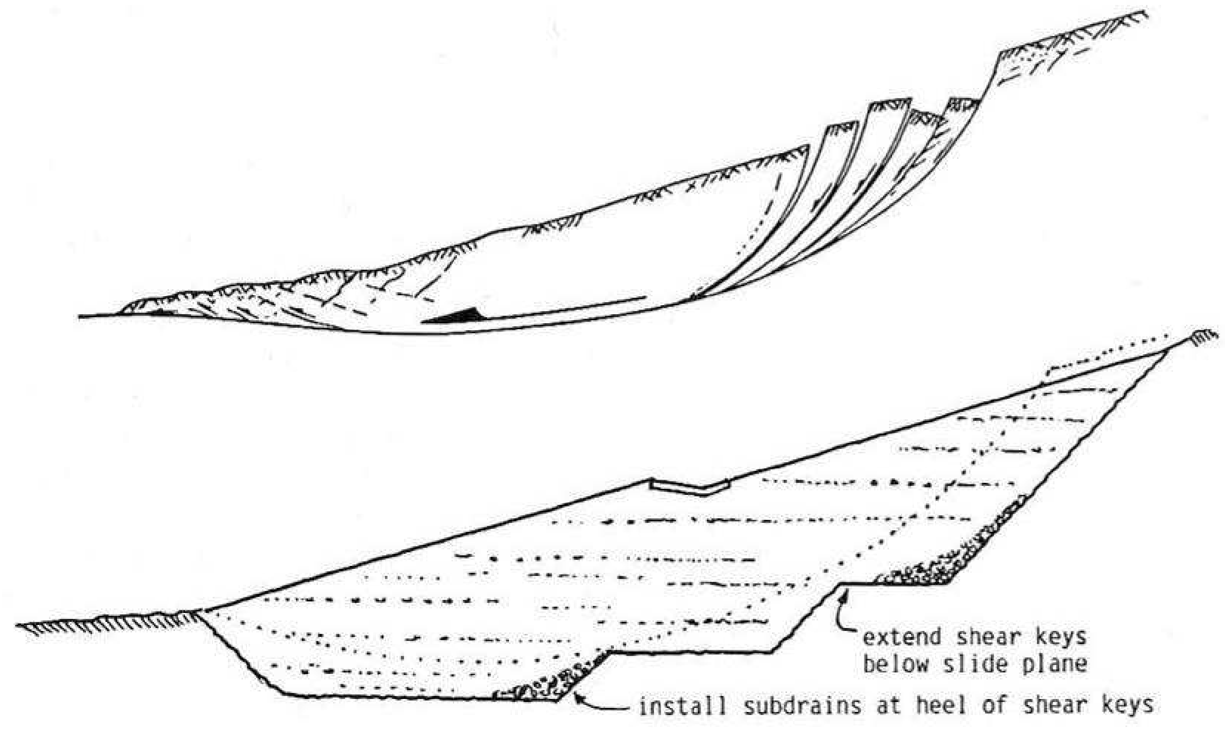
- "Shallow" substitution, where the unstable materials is partially or totally removed in bulk excavation and replaced with materials with adequate strength and permeability characteristics using standard earthworks equipment. Depending on the size and shape of the landslide, "shallow" substitution may involve from a few cubic metres to tens of thousand of cubic metres.
- "Deep" substitution, where conventional earthworks become impractical or uneconomic and substitution of unstable materials can be obtained by means of special techniques, typically vibro replacement/vibrodisplacement and jet-grouting.

"Shallow" substitution of a significant portion of the landslide by conventional earthworks is described here.
Partial "shallow" substitution to form drainage trenches or to form structural counterforts to transfer loads to stable layers below the landslide are discussed in Sections 4 and 6 of this Annex respectively.
Special techniques for deep substitution are described in the relevant fact sheets of Section 5.

Large scale excavation and replacement or recompaction of the landslide body have become feasible and progressively more widely practiced with the introduction of large self-propelled hydraulic-powered earthmoving and compaction equipment in following World War II (Rogers, 1992).
Where the volume of the sliding mass is relatively small and shallow, so that there is limited space for compaction, the excavated material is often replaced with crushed gravel or stone fill, which requires limted compaction and provides excellent drainage characteristics. The development of geosynthetics and in particular of filter fabrics has provided a fast and economic solution to the problem of migration of fines from the underlying fine fill or natural soil to the gravel or stone fill. However, considering that typically the shear strength at the soil-geotextile interface is lower than within the soil itself, it is necessary to step the base of the excavation to prevent forming an artificial discontinuity where further sliding can take place.
For larger slides the use of imported high quality fill is expensive and implies a significant environmental impact, not only associated with the quarrying and transportation of the imported fill but also with the disposal of the excavated material. Accordingly, in larger slides the excavated soil is normally used to backfill the slide, relying on a number of techniques to prevent further sliding, where required.
Drainage installed at the heel of the basal shear keys and drainage layers within and at the base of the backfill prevent future rises in pore water pressures in the backfill.
In clay, excavation and recompaction destroys the slip plane at the base of the slide, where only residual strength is available, and replaces it with homogeneous material; if necessary, the clay backfill can be improved by lime stabilization, both to ease handling and compaction and to improve its mechanical characteristics.
Additional reinforcement may be added to the backfill, effectively forming a reinforced soil structure, as described in greater detail in Section 7. This is especially useful where failure has occurred in very steep slopes, which cannot be reconstructed with standard fill.
Sisson (2010) describe a recent example of a major landslide repaired by substitution in Oceanside, California.

Design
Shallow substitution
For shallow substitution with unreinforced fill, the design process is the same as would be carried out for new fill, typically based on limit equilibrium analyses. Reference should be made to Section 2 of the Annex for further discussion on the applicability and limitations of these methods when used to evaluate "first time slides". Special care should be paid in ensuring the stability of temporary excavations.

Figure 1: Substitution of failed soil mass by excavation and recompaction (source: Rogers., 1992)



MITIGATION THROUGH REDUCTION OF HAZARD			
5 MODIFYING THE MECHANICAL CHARACTERISTICS OF UNSTABLE MASS			
5.2 SUBSTITUTION			
APPLICABILITY			
Class	**Descriptor**	**Rating**	**Notes**
Type of movement (Cruden & Varnes, 1996)	Falls	0	Best suited to rotational slides. Also suitable for translational slides, depending on geometry and extent.
	Topples	0	
	Slides	8	
	Spreads	0	
	Flows	0	
Material	Earth	8	Rock may imply difficulties in excavation and the need to form steep slopes,, which require some form of reinforcment.
	Debris	8	
	Rock	6	
Depth of movement	Superficial (< 0.5 m)	8	Very small, superficial slides may be repaired by granular fill with limited compaction. Larger slides (medium deep) allow conventional construction equipment to operate efficiently. Intermediate slides cannot be addressed efficiently in either way, while very large, deep landslide involve large earthmoving with significant potential environmental and cost impacts.
	Shallow (0.5 to 3 m)	6	
	Medium (3 to 8 m)	8	
	Deep (8 to 15 m)	4	
	Very deep (> 15 m)	0	
Rate of movement (Varnes, 1978)	Moderately to fast	0	While excavation can be carried out without special difficulty when the rate of movement is slow (5cm/day) or less, backfilling presupposes that the slide is stable or moving at most very slowly.
	Slow	2	
	Very slow	6	
	Extremely slow	10	
Groundwater	Artesian	2	High or artesian groundwater conditions pose special problems, both to the excavation and to the stability of the slope after backfilling, limiting the applicability of this techniques, when these conditions occur, unless the long term stability of the backfill is improved by combination with deep drainage.
	High	4	
	Low	8	
	Absent	10	
Surface water	Rain	8	Surface flows must be diverted to prevent them from accumulating in the backfill. Drainage to be provided both on surface and at interface between fill and natural soil.
	Snowmelt	8	
	Localized	8	
	Stream	2	
	Torrent	0	
	River	0	
Maturity		8	Concept an techniques well developed.
Reliability		8	The reliability of the technique depends on the evaluation of the stability of the treated slope.
Implementation		8	Can be implemented with widely available equipment. Possible difficulties with excavation in rock and with the disposal of arisings. Construction control.
Typical Cost		8	Low to moderate, depending on the material used (imported or from excavation).

Note
Ratings are given on a scale of 1 to 10; the higher the grade, the most suitable is the specific method under consideration to use in landslides of the given characteristics, evaluated individually. Overall suitability to specific case under consideration may be obtained by a weighted average of these ratings, with user defined weights. Zero rating means "not applicable"

MITIGATION THROUGH REDUCTION OF HAZARD

5 MODIFYING THE MECHANICAL CHARACTERISTICS OF UNSTABLE MASS

5.2 SUBSTITUTION

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MITIGATION THROUGH REDUCTION OF HAZARD

5 MODIFYING THE MECHANICAL CHARACTERISTICS OF UNSTABLE MASS

5.3 COMPACTION FROM SURFACE

Description

Compaction of the natural material may be carried out from the surface applying one of the following principles:

1. Compaction due to high static pressures from heavy equipment;
2. Compaction due to vibratory equipment;
3. Compaction due to heavy impact, using either eccentric rollers or dynamic compaction;
4. Compaction due to pressure waves induced by blasts.

All systems aim to cause the soil grains to rearrange into a denser microstructure (fabric); they are effective only on unsaturated materials, with relatively low water content or, to a lesser extent, in free draining materials where pore water can readily escape (Forssblad, 1981)

Impact compaction, dynamic compaction and blasting are not considered further, since they are not applicable to landslide stabilization because the very high levels of energy involved could itself trigger movement and because of the intrinsic difficulties of applying these techniques on sloping ground. In fact, even vibratory compaction needs to be applied with caution in certain conditions. There are reports of landslides in quick clay triggered by construction-induced vibrations. Vibration from compaction may also cause nuisance and in extreme cases damage outside the zone of application, to a distance of several tens of metres.

High pressure compaction

Surface compaction is generally achieved by driving heavy equipments repeatedly on the soil. Different types of equipment have been developed for this purpose.

The simplest equipment consists of a heavy duty machines or towed units with regular tires, referred to as “pneumatic rollers” (Figure 1a); these compactors may commonly be as heavy as 500 to 2000 kN. As these compactors run slowly on the ground, the top soil gets mechanically compacted by the temporary increased vertical stresses.

Other equipment, referred to as “sheep’s foot roller” (Figure 1c), has been designed to penetrate into the shallow soil to get better compaction; the penetrating parts result in a smaller contact surface to the soil and thus in higher pressures applied; pressures as high as 4.2 MPa may be achieved by the heaviest equipments in common use. The penetrating method is applicable only in presence of fine grained materials, resulting ineffective in coarse grained materials. In clay, these rollers prevent the formation of pre-sheared surfaces sub-parallel to the compaction surface, which can be highly deleterious to stability.

Vibratory compaction

Vibratory compactors are available with vibrating drums (Figure 1d), pneumatic tires or plates (Figure 1b). These compactors use high frequency, low amplitude vertical oscillations in addition to high vertical stresses due to their high weight. In this way the material is shaken and brougth into a more dense state.

As for the non-vibratory equipments, the smoot surface equipments are best suited to compact coarse grained materials; padded or “lagged” equipments, like a vibratory “*sheep’s foot roller*”, are best suited for fine grained materials.

Even in optimal conditions, with these methods the maximum thickness of improvement is less than 2 m and more often less than 0.5 to 1.0 m, hence the applicability of these methods to slope stabilization work is limited.

The high weigth of the equipments is also a limitation. Heavy rollers (static and vibratory) are designed to operate on quasi-level ground; they become relatively ineffective and difficult to operate on sloping ground. On relatively short slopes they can operate along the line of maximum slope assisted by a winch securely anchored at the top of the slope, but this severely limits their operation and may have safety implications.

Design

Compaction of the top 0.5 to 1.0 m of soil should be sufficient to produce density states characterized by strong interlocking of grains, making the material highly dilatants and thus resistant to shear stresses and the erosive effects of wind, rain and runoff. Compaction can be specified in terms of “method”, detailing the type of equipment and the compaction procedure to be adopted, or in terms of “performance”, specifying the density to be achieved. This is typically specified in terms of the dry density to be achieved in relation to standard (for fine grained soils) or modified (for granular soils) Proctor compaction tests. For granular soils it is also common to refer to relative density instead (Parsons, 1987).

Figure 1: Different surface compaction equipment:
a) Pneumatic compactor (source http://kudat68.en.made-in-china.com/);
b) Backhoe-attached vibratory plate compactor (source www.construction-int.com)
c) Sheep's foot drum, pulled unit (source www.youngsweldinginc.com)
d) Vibratory smooth drum compactor (source: www.fhwa.dot.gov)

MITIGATION THROUGH REDUCTION OF HAZARD

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5.3 COMPACTION FROM SURFACE

APPLICABILITY

Class	Descriptor	Rating	Notes
Type of movement (Cruden & Varnes, 1996)	Falls	0	Only possibly suitable for shallow translational or very small circular slides; can improve erosion resistance of loose graded soils.
	Topples	0	
	Slides	4	
	Spreads	0	
	Flows	0	
Material	Earth	6	Applicable in fine to coarse soil and small debris. Ineffective on corse debris and rock.
	Debris	4	
	Rock	0	
Depth of movement	Superficial (< 0.5 m)	6	Maximum depth of effectiveness typically 0.5 to 1.0 m.
	Shallow (0.5 to 3 m)	2	
	Medium (3 to 8 m)	0	
	Deep (8 to 15 m)	0	
	Very deep (> 15 m)	0	
Rate of movement (Varnes, 1978)	Moderately to fast	0	Surface compaction presupposes that the slide is stable or moving at most very slowly.
	Slow	0	
	Very slow	2	
	Extremely slow	8	
Groundwater	Artesian	0	Ineffective on saturated soil, unless free draining.
	High	2	
	Low	8	
	Absent	8	
Surface water	Rain	6	Can improve resistance to soil to erosion by rain and runoff.
	Snowmelt	6	
	Localized	2	
	Stream	0	
	Torrent	0	
	River	0	
Maturity		6	Applicability of shallow compaction as a slope stabilization techniquie unproven.
Reliability		4	Effectiveness of compaction on slope to be confirmed on a case by case basis.
Implementation		8	Significant difficulties operating heavy compaction equipment on slopes. Vibrating plates mounted on booms have limited reach.
Typical Cost		8	Low.

Note
Ratings are given on a scale of 1 to 10; the higher the grade, the most suitable is the specific method under consideration to use in landslides of the given characteristics, evaluated individually. Overall suitability to specific case under consideration may be obtained by a weighted average of these ratings, with user defined weights. Zero rating means "not applicable"

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5.3 COMPACTION FROM SURFACE

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5 MODIFYING THE MECHANICAL CHARACTERISTICS OF UNSTABLE MASS

5.4 DEEP COMPACTION (VIBROCOMPACTION – VIBROREPLACEMENT - VIBRODISPLACEMENT)

Description

Vibrocompaction

The vibrocompaction technique, also known as vibroflotation, is suitable for compacting thick layers of loose granular deposits (gravels, sands). The maximum depth of compaction is typically limited by the lifting equipment. Depths up to 70 metres have been achieved (Moseley and Kirsch, 2004; www.vibroflotation-ng.com).

Deep compaction is normally achieved according to the following steps (Figure 1.):

1. A probe is penetrated to the desired depth under its own weight with minimal vibration and with the assistance of high pressure water jet from the tip of the probe, progressively displacing the soil beneath it (Figures 2 and 3).
2. At the desired depth the vibroprobe is activated, oscillating laterally and transferring vibrations horizontally into the surrounding soil, compacting it. The area of influence depends on several factors, mainly the mechanical characteristics of the vibroprobe, the target relative density to be achieved, the nature of the soil and groundwater levels. Guidance on what can be achieved with standard equipment is provided by Elias et al. (2001). A spacing of 3 m is typically adopted when using standard equipment in favourable conditions .
3. The vibroprobe is slowly raised towards the surface while vibrating. The overlaying soil will gradually sink in, as the lowermost material is densified (Figure 4). Additional sand is usually dropped into the hole to ensure full compaction of the area.

Deep compaction due to vibration may also be achieved penetrating a hollow steel tube into the soil, a method referred to as *Terra-Probe*; the steel tube is vibrated down to the desired depth and then drawn up again while the hollow steel tube is vibrating; this procedure is repeated several times to get the required degree of compaction. In this technique the vibrator is mounted on the top of the stell tube and imparts vertical, rather than horizontal vibration, resulting in a much smaller area of influence. A spacing of 1.5 m is typically adopted when using the *Terra-Probe* equipment in favourable conditions; the greater quantities are compensated in part by the greater speed compared to vibrofloatation.

Notwithstanding the addition of material at each treatment point during compaction, in both cases settlement is normally induced by vibration, which can be compensated either by overfilling with clean granular soil prior to compaction or by conventional filling and compaction at the end of treatment, if required.

According, for example to Bergado et al. (1999) and Mc Carthy (2007), clay and silty content should be less than 15 to 20% for the method to be effective. Higher contents of silt and clay will limit the ability of water to drain away rapidly and may result in the sides of the hole not “collapsing” promptly onto the probe, reducing the effectivness of energy transfer from the probe to the surrounding soil, thus limiting the compaction process. Gravel content should be less than 20%; higher gravel contents may limit the ability of the probe to penetrate the soil to be compacted, thus limiting the maximum depth of treatment to, say, 10 m depth.

Great caution is necessary when performing deep compaction near existing services or structures, to limit settlements, and below the groundwater level, to limit the resulting excess pore pressures not to trigger local or general instability.

Provided vibrocompaction is carried out properly and with the appropriate spacing between treatment points, the treated soil may be considered as a continuous medium with improved and more homogeneous mechanical characteristics; in particular, as a consequence of the increase of density, both stiffness and strength are increased.

Stone columns

Stone colums consist of underground colums of crushed rock or gravel, installed by techniques similar to those adopted and described above for vibrocompaction. Stone columns are adopted where vibrocompaction ceases to be effective, e.g. where silt and clay content is higher than 15 to 20%. Two different methods can be adopted to form stone columns, e,g, vibroreplacement and vibrodisplacement. In both cases, stone columns reinforce all the layers crossed, including un-compactable layers. Depending on the nature of the soil and the particulars of the technique used, their installation may also result in the compaction of the original soil between columns.

Vibroreplacement can be carried out using either a wet process (water jet) or a dry process (air jet); normally the wet method is more effective. Besides the vibroprobe and the supporting crane, which are essentially the same used for vibrocompaction, the spread of equipment includes a compressor and a wheel loader (Figure 6).

Figure 1: The process of vibrocompaction (source: www.vibroflotation-ng.com)

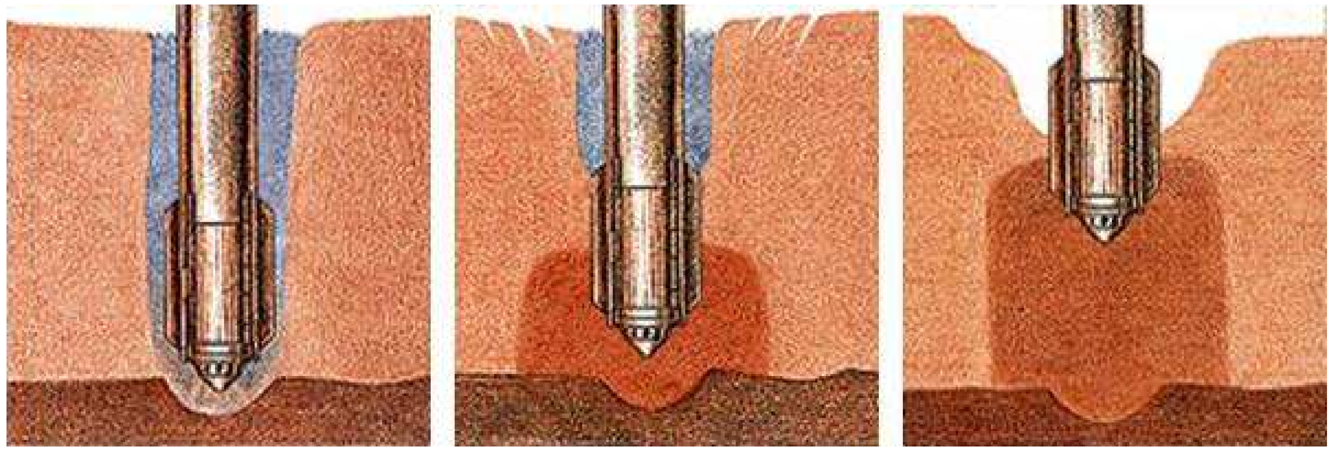

Figure 2: Typical equipment for vibro compaction (source: SGI-MI project files)

Figure 3: Typical equipment for vibro compaction (source: SGI-MI project files)

Figure 4: Surface crater during vibro compaction (source. www.kellergrundbau.com)

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5 MODIFYING THE MECHANICAL CHARACTERISTICS OF UNSTABLE MASS

5.4 DEEP COMPACTION (VIBROCOMPACTION – VIBROREPLACEMENT - VIBRODISPLACEMENT)

The vibroprobe, which is the only specialist equipment, can be easily transported by container and assembled on site (Figure 7), while the rest of the equipment can be hired locally. The technique consist of the following steps (Figure 5):

1. As in the case of vibrocompaction, the probe is lowered to the desired depth. In silty soils the fines are washed to the surface by water circulation (Figure 8); the washings need to be collected and disposed of in a controlled manner.
2. The probe is then lifted up a short distance (e.g. 0.5 m) and a backfill of stone is introduced in the hole from the top (Figure 9). The added material is then repenetrated by the vibroprobe, which compacts it and pushes it against the surrounding soils, ensuring good contact and energy transfer between the probe and the surrounding soil, increasing the width of the stone columns.
3. The procedure described at point 2 is repeated until the stone column reaches the surface.

Vibrodisplacement is performed dry (air jet) according to the following steps (Figure 10):

1. As in the case of vibrocompaction and vibroreplacement, the probe is lowered to the desired depth. Contrary to vibroreplacement, the use of air jets only precludes the washing out of fines and all the soil is displaced laterally. However, this results in a greater resistance to penetration and "preloosening" may be required, especially if local dense layers exist above the layers to treated. This can be carried out by inserting a continuous flight auger and retrieving it by counterrotation without soil removal (Figure 13).
2. The probe is then lifted up a short distance (e.g. 0.5 m) and gravel loaded in an airlock chamber is delivered to the bottom through the vibroprobe or a separate pipe (Figures 11 and 12). The grading must be carefully controlled to avoid blockage of the delivery pipe. The added material is then repenetrated by the vibroprobe, which compacts it and pushes it against the surrounding soils compenetrating or displacing it, ensuring good contact and energy transfer between the probe and the surrounding soil, increasing the width of the stone columns and inducing further densification/compaction of the soil between columns.
3. The probe is gradually lifted in stages, continuously adding and compacting coarse material as described at point 2. More material will be added where soil is weaker.

The methods should not be used in saturated soft sensitive clays as the vibration and pressures from the stone columns on the surrounding soil may exceed its strength and destabilize the slope (Ground Improvement Solutions, 2010)

Great caution is necessary when performing deep compaction near existing services or structures, to limit settlements and horizontal displacements, and below the groundwater level, to limit the resulting excess pore pressures not to trigger local or general instability. The use of compressed air may also have undesirable side effects in certain circumstances.

Stone columns increase stability through all soil layers because of higher shear strength of the coarse fill material; their installation may also improve the mechanical characteristics of the soil between columns, especially if the vibrodisplacement method is used; in certain conditions they may also improve drainage, provided a suitable outfall exists or is provided.

Design

In general all the methods described above are applicable in saturated relatively coarse grained materials (gravels, sands, sandy silts) susceptible to liquefaction related phenomena induced by monotonic or cyclic (vibration, earthquake, waves, etc.) stress changes.

Stone columns may be used also to improve the composite shear strength of a deposit, but this may be better achieved by other methods unless it is also possible and necessary to mobilize their potential drainage effect.

Vibrocompaction

The degree of compaction to be reached by vibrocompaction should be determined in terms of achieving a significant reduction in the susceptibility of the soil to develop of excess pore pressures under monotonic or cyclic loading. This presupposes a detailed understanding of the triggering mechanisms. The following general considerations apply:

- Under static loading, the density of the material must be sufficient to preclude the occurrence of stress states located on or above the Collapse /Instability Surface (Sladen et al., 1985; Ishihara, 1993; Lade, 1992; Lade, 1993) or, more in general within the Instability Zone (Lade and Pradel, 1990; Leong et al., 2000; Chu et al., 2003), where flow-type instability could be triggered. Examples of how the density of the materials affects the position of the Instability Zone and hence the stability of the slope are presented and discussed in the Deliverable 1.1 of the SAFELAND Project., where the terminology used here is also explained in detail

Figure 5: The process of vibroreplacement (source: www.vibroflotation-ng.com)

Figure 6: Vibroreplacement equipment: crane, vibroprobe, compressor, wheel loader (source: SGI-MI project files)

Figure 8: Penetration of probe in silty sand; note fines washed out by water circulation (source: SGI-MI project files)

Figure 7: Specialist vibroreplacement equipment can be transported in containers(source: SGI-MI project files)

Figure 9: Stone added to top of column by weel loader during alternate movement of the probe (source: SGI-MI project files)

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5 MODIFYING THE MECHANICAL CHARACTERISTICS OF UNSTABLE MASS

5.4 DEEP COMPACTION (VIBROCOMPACTION – VIBROREPLACEMENT - VIBRODISPLACEMENT)

The evaluation of the potential for flow-type instability in sandy materials can be made on the basis of simplified procedures which use the results of SPT and/or CPT tests (see for example Ishihara, 1993; Fear and Robertson, 1995; Cubrinowski and Ishihara, 2000; Olson and Stark, 2003a). Alternatively, a comprehensive program of laboratory tests on both "undisturbed" and reconstituted samples should be carried out to determine the Steady State Line and the position of the in situ state of the material referred to this line (see for example Been & Jefferies, 1985; Boulanger, 2003). In fact, it has been recognized that flow-type instability may occur only where penetration resistances are lower than appropriately defined threshold values and/or the initial states are located slightly below the Steady State Line.

- Under seismically induced cyclic loading, the density of the material should be sufficient to limit the development of excess pore water pressures; considering the short duration of seismic motion, reference may be made to "fully" undrained conditions. The verifications may be carried out as follows:
  - Step 1. Evaluation by the "simplified" method originally developed by H.B. Seed and coworkers of the susceptibility to triggering of seismic liquefaction, taking into account the effect of static shear stress by the coefficient K_α (see for example Idriss and Boulanger, 2008), to determine Factors of Safety against liquefaction F_L at different depths. Evaluation of the seismically induced excess pore water pressures as indicated, for example, by Seed et al. (1976), Ishihara and Nagase (1980), Finn (1981), Marcuson et al. (1990), Idriss and Boulanger (2008).
  - Step 2. Evaluation of slope stability using limit equilibrium methods and an equivalent pseudo-static action to model the earthquake loads. The analyses must be carried out in undrained conditions in terms of effective stresses (UES) and/or in terms of total stresses (UTS). The UES conditions will be considered in layers where the analyses of liquefaction potential have given safety factors everywhere higher than 1; the amount of excess pore pressures to be considered in calculation will be determined from step 1. The UTS conditions will be considered in layers where the liquefaction potential analyses have given safety factors equal to or less than 1; the undrained shear resistances to be considered in these layers may be determined according to the recommendations given by Olson and Stark (2002), Olson and Stark (2003a), Olson and Stark (2003b) and Mesri (2007).
  - 2D or 3D numerical dynamic analyses should be carried out as a final check, and in any case where it is necessary to estimate the seismically induced displacements. These analyses should be carried out in the time domain in undrained conditions using advanced costitutive models (see for example Manzari and Dafalias, 1997; Li and Dafalias, 2000; Li, 2002) capable of replicating the monotonic and cyclic soil behaviour measured in laboratory tests on "undisturbed" and reconstituted samples.
- For under water slopes and for wave induced cyclic loading (see for example Madsen, 1978; Okusa, 1985; Magda et al., 1994; Sassa and Sekiguchi, 1999; Sassa and Sekiguchi, 2001; Sassa et al., 2001), the density of the material should be sufficient to limit the development of excess pore water pressures. Considering the typical frequency of waves and the duration of storms, the development of excess pore pressures occurs under conditions of partial drainage, requiring 2D or 3D numerical dynamic analyses carried out in the time domain in conditions of coupled consolidation using advanced costitutive models as described above for the earthquake case.

Tests should be carried out after the treatment to verify that the required density has been reached.

Stone columns

Stone columns are inclusions of highly compacted stone or gravel with excellent mechanical characteristics and high permeability which act both as reinforcement and as drainage elements which favour the dissipation of excess pore pressures.

Verifying the effectiveness of stone columns is much more complex compared to vibrocompaction, since it involves the behavior of a dishomogeneous and discontinuous medium, thus necessarily requiring some gross simplifications.

The simplified methods currently available are based on limt equilibrium methods and on the following assumptions:

- Stone columns are sufficiently free draining to be immune from excess pore pressures; they can be modelled in terms of drained strength parameters under all loading/environmental conditions.
- The surrounding soil can be modelled in terms of the least of its drained and its undrained strength; the latter may be evaluated on the basis of empirical correlations as proposed, for example, by Olson and Stark (2002), Olson and Stark (2003a), Olson and Stark (2003b) and Mesri (2007) at pre-liquefaction and post-liquefaction conditions.

The improvement of the natural soil due to the installation of stone columns is normally ignored unless proven and quantified by appropriate full scale field tests

Figure 10: The process of vibrordisplacement (source: www.vibroflotation-ng.com)

Figure 11: Vibrodisplacement equipment: crane, vibroprobe with parallel gravel pipe and airlock chamber, loading skip, compressor and wheel loader (source: SGI-MI project files)

Figure 12: Probe with separate gravel delivery pipe and air nozzles for vibrodisplacement (source: SGI-MI project files)

Figure 13: "Preloosening" may be carried out by continuous flight auger, without soil extraction (source: SGI-MI project files)

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5 MODIFYING THE MECHANICAL CHARACTERISTICS OF UNSTABLE MASS			
5.4 DEEP COMPACTION (VIBROCOMPACTION – VIBROREPLACEMENT - VIBRODISPLACEMENT)			
APPLICABILITY			
Class	**Descriptor**	**Rating**	**Notes**
Type of movement (Cruden & Varnes, 1996)	Falls	0	Applicable to rotational and translational slides. In particular circumstances it may be applicable to spreads and flows.
	Topples	0	
	Slides	6	
	Spreads	4	
	Flows	4	
Material	Earth	8	Possible difficulties penetrating coarse debris.
	Debris	4	
	Rock	0	
Depth of movement	Superficial (< 0.5 m)	0	Best suited to medium to deep compaction. Uneconomic for shallow depths.
	Shallow (0.5 to 3 m)	0	
	Medium (3 to 8 m)	8	
	Deep (8 to 15 m)	8	
	Very deep (> 15 m)	6	
Rate of movement (Varnes, 1978)	Moderately to fast	0	Treatment presupposes that the slide is stable or moving at most very slowly.
	Slow	0	
	Very slow	2	
	Extremely slow	8	
Groundwater	Artesian	0	Technique potentially applicable but possibly unnecessary with low or absent groundwater levels.
	High	8	
	Low	6	
	Absent	6	
Surface water	Rain	8	Water courses must be diverted from treatment area.
	Snowmelt	8	
	Localized	8	
	Stream	2	
	Torrent	0	
	River	0	
Maturity		6	Limited experience of application to slope stabilization onshore. More widely used for preventive stabilization of marine slopes.
Reliability		8	Well developed technology. Reliable where applicable.
Implementation		6	Requires specialist equipment and know-how. Crane suspended equipment requires stable working platform and poses potential safety problems.
Typical Cost		4	Moderate to high, depending on whether imported stone/gravel is used and transport distance.

Note
Ratings are given on a scale of 1 to 10; the higher the grade, the most suitable is the specific method under consideration to use in landslides of the given characteristics, evaluated individually. Overall suitability to specific case under consideration may be obtained by a weighted average of these ratings, with user defined weights. Zero rating means "not applicable"

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5.4 DEEP COMPACTION (VIBROCOMPACTION – VIBROREPLACEMENT - VIBRODISPLACEMENT)

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5 MODIFYING THE MECHANICAL CHARACTERISTICS OF UNSTABLE MASS

5.5 MECHANICAL DEEP MIXING WITH LIME AND/OR CEMENT

Description

Mechanical deep mixing is the creation of vertical inclusions (columns or barrettes) by blending in-situ the soil with a stabilizing admixture to improve its mechanical characteristics (higher strength, lower compressibility). It is typically performed by specialist rotary equipment with mixing blades, which are inserted into and removed from the ground nominally without soil extraction while the admixture is injected from nozzles in or near the blades (Figure 1). The hydraulic conductivity of the treated soil will be higher or lower than that of the parent soil, depending on the soil type and admixture used. The admixture consists of stabilizing binders that react chemically with water, resulting in cation exchange on the surface of clay minerals or bonding of soil particles and/or filling of voids (Terashi, 2003). The most common binders are cement or lime; other materials like gypsum or fly ash are also used (Moseley and Kirsch, 2004).

The method is best suited to soft fine grained materials of relatively low shear strength and is applicable down to a depth of 30 m. The effect on slope stability depends on the type of soil being treated, the layout and spacing of the inclusions, the type of admixture used and the equipment and method of mixing (Mc Carthy, 2007).

Research and development of deep mixing as it is known today started in Japan and in Sweden in the late 1960's using blades rotated by a single vertical shaft and lime as a binder, with the first applications being impleneted in the mid 1970's. Since then there have been significant developments in all aspects of the technology.

Different equipment and procedures have been developed to respond to different soil conditions and performance requirements. Figures 2 and 3 show typical equipment developed in Scandinavia, consisting of relatively lightweight rigs and trailers loaded with dry binder. This equipment is suitable for treating extremely soft, "quick" soil, which can be mixed satisfactorily with very lightweight, fixed blades (Figure 4); dry binder is used, reacting with the soil pore water. Heavier, stiffer soils, possibly mixed with silt or even granular soils, require more robust, heavier equipment with a mix of rotating and fized or even counter-rotating blades to break up lumps (Figure 5). A thick slurry of binder is normally used in these soils (wet method). The selection of dry versus wet soil mixing is normally made on the basis of the natural water content and undrained shear strength of the natural soil, dry mixing being preferred where the natural moisture content of the soil is greater than 60% and its undrained strength less than 70-75 kPa.

A further development in single shaft technology has been the introduction of composite systems which combine deep mixing and jet grouting techniques. The jet grouting nozzles are located on the outer edge of the mixing blade (Figure 6) such that the completed column has a mechanically mixed core and a jet grouted annulus (Figure 7).

Multi-rotary equipment (Figure 8) has been developed primarily to allow simultaneous installation of 2 or more secant circular columns to form wall panels of mechanically mixed stabilized soil for the construction of temporary or permanent walls. These systems have the added benefit that mixing is much enhanced by the action of counter-rotating blades on adjacent, compenetrating, columns (Figure 9). An additional benefit specific to landslide mitigation or remediation is that panels are much better than isolated columns in resisting landslide loads, as discussed in fact sheet 5.0.

The need to form panels has driven the development of radically different approaches, deviating from the technology based on blades rotating around the vertical axis. Discrete panels or barrettes can be formed by two cutter/mixer heads counter rotating around horizontal axes (Figures 10, 11 and 12). Continuous walls can be formed, but only to a limited depth, by a continuous chain cutter/mixer (Figures 13and 14). In all cases the dimensions of the resulting inclusions are the same as those of the mixer (auger or cutter).

The method may be applicable with caution to sensitive clay since probably the installation process does not induce significant pressure in the surrounding material and the temporary change in slope stability may probably be disregarded.

Design

Unless mass treatment is carried out, which is highly unusual, the verification of effectivness of the treatment is complex, since it refers to the behaviour of a discontinuous mass. It can only be addressed by applying significant simplifications. Available simplified methods are based on limit equilibrium (in static and seismic conditions):

The properties of the inclusions are pre-determined from laboratory tests carried out at different confining pressures to determine the strength envelope of the treated soil in terms of both total and effective stress. Bearing in mind that due to inmperfect mixing filed strengths are typically only 35 to 50% of the strength measured in laboratory tests, the actual strength of the treated soil needs to be verified by trial fields and control tests.

The surrounding (clay) soil can be modelled in terms of undrained shear strength, with appropriate reductions in case of cyclic loads (see for example Idriss and Boulanger, 2008).

Figure 1: Principle of deep soil mixing, here; dry soil mixing (source: McCarthy, 2007)

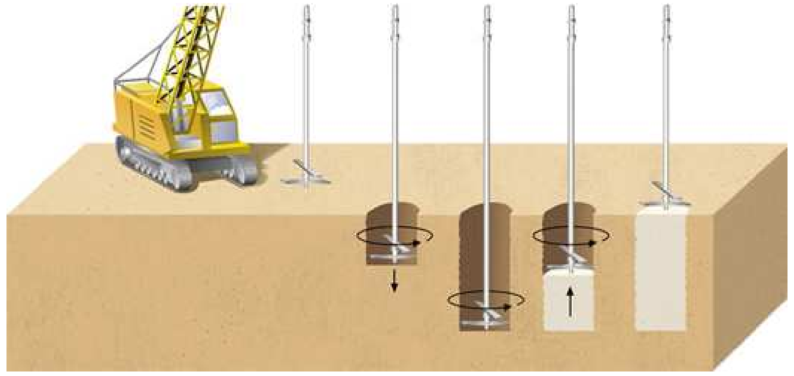


Figure 2: Principle of deep soil mixing, here; dry soil mixing (source: McCarthy, 2007)

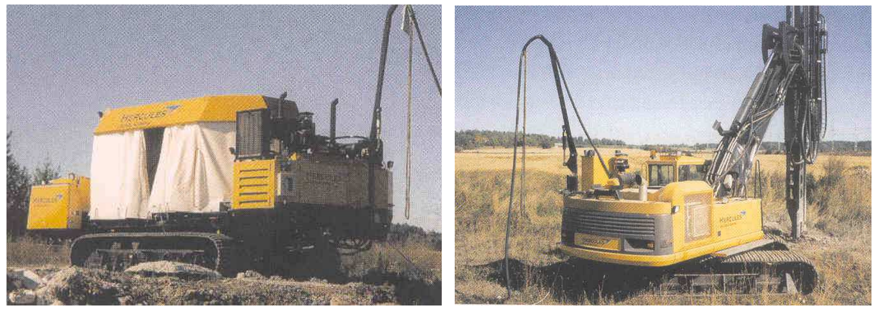


Figure 3: Principle of deep soil mixing, here; dry soil mixing (source: McCarthy, 2007)

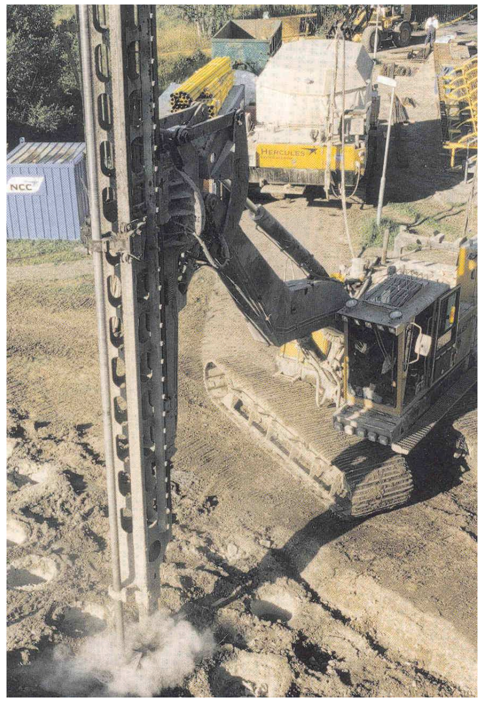


Figure 4: Typical mixing tool with light blades developed in Scandinavia for mixing very soft, sensitive clay (source: Hercules Grundläggning)

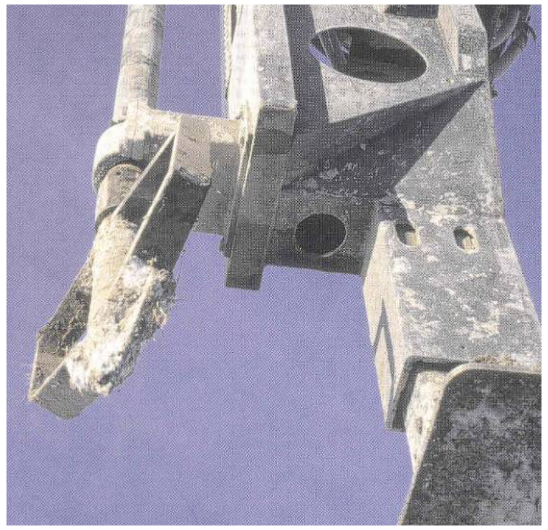


Figure 5: Typical tool with complex blade arrangement developed in Japan for mixing soft to firm clays and silty clays (source: Hayward Baker)

MITIGATION THROUGH REDUCTION OF HAZARD

5 MODIFYING THE MECHANICAL CHARACTERISTICS OF UNSTABLE MASS

5.5 MECHANICAL DEEP MIXING WITH LIME AND/OR CEMENT

Figure 6: Tool for composite mechanical mixing and jet groutinged (source: Terashi, 2003)

Figure 8: Equipment for simultaneous installation of multiple columns (source: Schnabel)

Figure 10: Equipment for mechanical mixing by horizontal drums (source: Soletanche Bachy)

Figure 11: Principle of deep soil mixing by counterroating horizontal cutter/mixer drums (source: Soletanche Bachy)

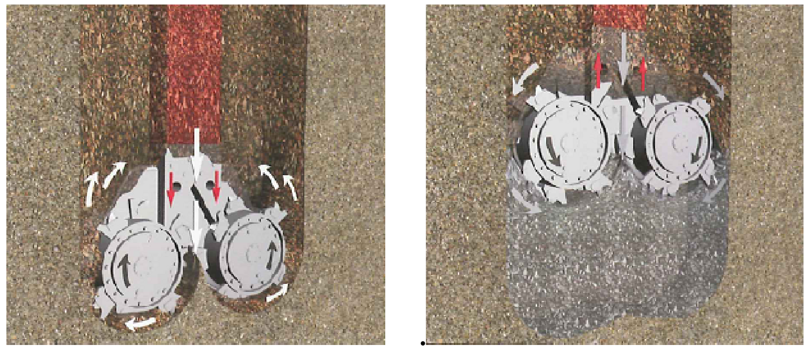


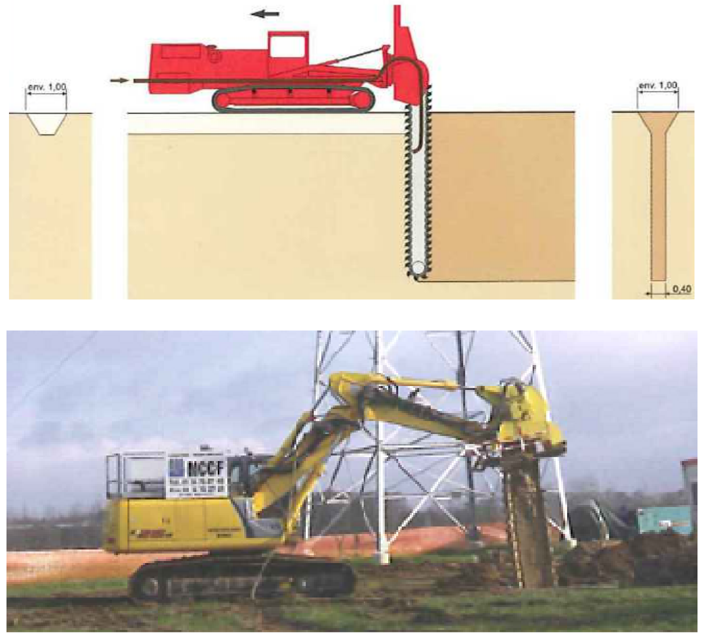
Figure 12: detail of cutter/mixer drums (source: Soletanche Bachy)

Figure 7: Composite column formed by mechanical mixing and jet groutinged (source: Terashi, 2003)

Figure 9: Equipment for simultaneous installation of multiple columns - detail (source: Schnabel)

Figure 13: Continuous panel soil mixing - Detail (source: Soletanche Bachy)

Figure 14: Principle and equipment of continuous panel soil mixing by continuous chain. Wet method illustrated here. (source: Soletanche Bachy)



MITIGATION THROUGH REDUCTION OF HAZARD

5 MODIFYING THE MECHANICAL CHARACTERISTICS OF UNSTABLE MASS

5.5 MECHANICAL DEEP MIXING WITH LIME AND/OR CEMENT

APPLICABILITY

Class	Descriptor	Rating	Notes
Type of movement (Cruden & Varnes, 1996)	Falls	0	Application to landslide stabilization generally mimited by need to use relatively heavy equipment. Applicability to spreads and flows to be carefully evaluated on a case by case basis, bearing in mind the risk that installation iteself could trigger movement
	Topples	0	
	Slides	6	
	Spreads	4	
	Flows	4	
Material	Earth	8	Most suited to fine soils. Not applicable in coarse debris and rock
	Debris	4	
	Rock	0	
Depth of movement	Superficial (< 0.5 m)	0	Typically inappropriate in shallow applications. The entire soil thickness needs to be treated, which makes it unsuitable for selective tretament at depth.
	Shallow (0.5 to 3 m)	4	
	Medium (3 to 8 m)	8	
	Deep (8 to 15 m)	8	
	Very deep (> 15 m)	6	
Rate of movement (Varnes, 1978)	Moderately to fast	0	Workers' safety and end result require construction to take place when movement is extremely slow or very slow (maximum 1.5 m/year or 5 mm/day). Under special conditions and taking due precautions, it may be carried out when movement is "slow" (up to 1.5 m/month, corresponding to 5 cm/day) .
	Slow	2	
	Very slow	6	
	Extremely slow	8	
Groundwater	Artesian	6	The possibility to operate with a dry binder or a slurry depending on conditions and the fact that the soil is never removed make the technique generally applicable in all groundwater conditions. Severe artesian groundwater conditions or strong underground flows may cause seepage induced leaching of the inclusion before the binder sets.
	High	8	
	Low	8	
	Absent	8	
Surface water	Rain	8	Water courses need to be temporarily diverted or reliably dry during construction. Potential pollution of watercourses during construction (for example by spillage of slurry) may impose restriction on construction procedure. No problems once the works are completed, except possibly when the inclusions provide an undesired "hard bank" to watercourses.
	Snowmelt	8	
	Localized	8	
	Stream	2	
	Torrent	2	
	River	2	
Maturity		6	The technique is well established and widely used for the preventive stabilization of engineering slopes; less so in the mitigation of natural landslides.
Reliability		8	Reliable performance in well characterized landslides; in first time slides it depends on estimate of piezometric regime and apprporiate operational strength parameters of soil, which can be problematic; problems may occur during construction, for example if unforeseen boulders are encountered.
Implementation		6	Requires specialist equipment and techniques; implementation may need temporary roads and working platform for safe operation.
Typical Cost		4	Relatively expensive.

Note
Ratings are given on a scale of 1 to 10; the higher the grade, the most suitable is the specific method under consideration to use in landslides of the given characteristics, evaluated individually. Overall suitability to specific case under consideration may be obtained by a weighted average of these ratings, with user defined weights. Zero rating means "not applicable"

MITIGATION THROUGH REDUCTION OF HAZARD

5 MODIFYING THE MECHANICAL CHARACTERISTICS OF UNSTABLE MASS

5.5 MECHANICAL DEEP MIXING WITH LIME AND/OR CEMENT

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MITIGATION THROUGH REDUCTION OF HAZARD

5 MODIFYING THE MECHANICAL CHARACTERISTICS OF UNSTABLE MASS

5.6 LOW PRESSURE GROUTING WITH CEMENTITIOUS OR CHEMICAL BINDER

Description

Grouting consistis of the injection of pumpable material into soil or rock under pressure through vertical or inclined boreholes, typically to a maximum depth of 50 m. Depending on the method of injection, grouting can be classified as slurry (intrusion) and permeation (penetration) grouting, where disturbance to the original soil structure is minimized (Figure 1), and displacement (compaction) grouting; jet grouting and fracture grouting, which deliberately disturb the original soil structure (Townsend and Anderson, 2004; Warner, 2004).

Slurry grouting (injection of flowable suspensions of cement/clay grouts into open cracks, fissures and voids) and permeation grouting (filling pore spaces in soil and joints in rock) are described here generally as "grouting". Jet grouting is described in fact sheet 5.7, while displacement and fracture grouting are not generally applicable to slope stabilization.

The most common grout materials are cement, microfine cement, lime, gypsum, sodium silicate chemicals and polymers (Warner, 2004; Mc Carthy, 2007). Different grout materials have different viscosity; the more viscous materials, such as cement grouts, are used for coarse grained soil and rock masses with open fractures; the less viscous materials, such as the chemical grouts, are used for fine grained materials (Figure 2). Indicatively, Ordinary Portland cement may be used in soils with D10 > 0.6 to 1.0 mm, while microfine cements may be used in soils with D10 > 0.08 to 0.1 mm. (Mitchell, 1981; Townsend and Anderson, 2004). Chemical grouts may be used in even finer soils. The most common chemical grout used for structural applications is sodium silicate. Other chemical grouts are acrylates and polyurethanes. In 1997 a release of acrylamide into the groundwater caused serious environmental problems in the area of the Hallandas Tunnel, near Baastad in Sweden (Lofstedt, 1999; Littlejhon, 2003), leading to the withdrawal of this chemical from the market. Since then the materials used in chemical grouts come under very close scrutiny for their potential environmental effects.

The process of injection is usually done by a movable injection rig according to the following steps and the same procedures apply for all methods of grouting and injection material:

1. An injection pipe is inserted into the ground to the required depth, either by static pressure (in loose soils) or more commonly by lowering it into a predrilled hole. Drilling is normally carried out by rotary/percussive or more commonly by percussive methods. Careful consideration is required in the selection of the appropriate flushing medium.
2. Typically the grout is injected from the end of the injection pipe while the pipe is withdrawn, either continuously or in predetermined discrete intervals for the full thickness of interest, resulting in vertical or inclined continuous columns of soil with improved characteristics, e.g. increased stiffness and strength and reduced permeability.
3. The procedure is carried out in several holes, usually in a close grid pattern. If the injection grid is made with small spacing the ground treatment may becomes "continuous" also in the horizontal direction.

Alternative procedures include injection through a pipe in an open hole sealed at the surface or through a grout pipe left in place as "tube a' manchette", although the latter is seldom used for low pressure grouting in stabilization projects, where the geometry of the grouted mass is not critical.

To ensure that the injections do not disturb the in-situ structure of the soil, special care is required in adjusting injection rates and pressures, as too high rates and pressures may displace grains or even worse result in hydraulic fracturing.

To confirm the geometry and effectiveness of treatment, injections rates, pressures and volumes must be accurately monitored and recorded, in association with corings for inspection and testing of the treated soil.

Grouting can be used to stabilize rock masses (Figure 3), for selective treatment of weak soil layers at depth or for stabilizing coarse grained soils susceptible to liquefaction related phenomena When grouting is carried out in slopes, drainage must be provided to avoid build up of pore water pressures behind the treated area.

Design

The true cohesion given by the treatment must be sufficiently high to resist the static and seismic loads without damage; in this case excess pore pressures may be considered negligible. In static conditions, the analyses may be carried out using limit equilibrium or FEM methods. In seismic conditions they may be carried out by limit equilibrium methods, modelling the seismic actions pseudo-statically, or by dynamic FEM methods in the time domain. The mechanical properties of the treated soil may be estimated initially from laboratory tests on samples compacted to the in situ density and permeated with the selected binder in the laboratory. These initial estimates will then need to be validated by laboratory tests on undisturbed samples of treated soil. The tests should be carried out at different confining pressures to determine the strength envelope in terms of effective stress. For preliminary estimates, unconfined compressive strengths of cement grouted soil typically range between 0.35 and 0.7 MPa, occasionally up to 2.0 MPa

Figure 1: Conceptual diagram of permeation grouting (source: Andrus and Chung,1995)

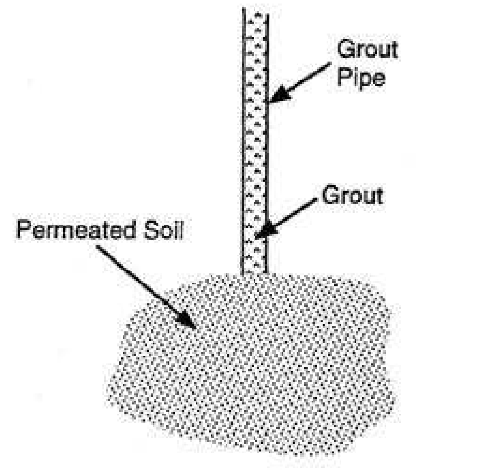


Figure 2: Soil-grout material compatibility (source: Townsend and Anderson, 2004)

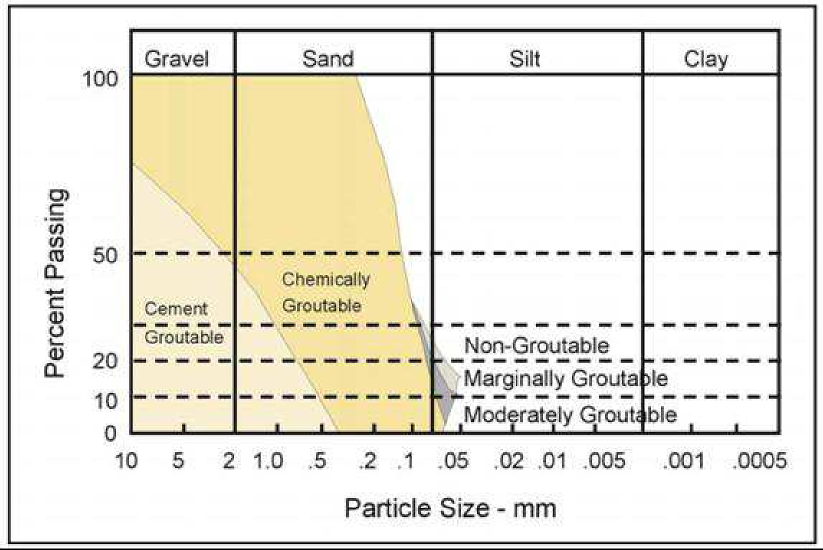
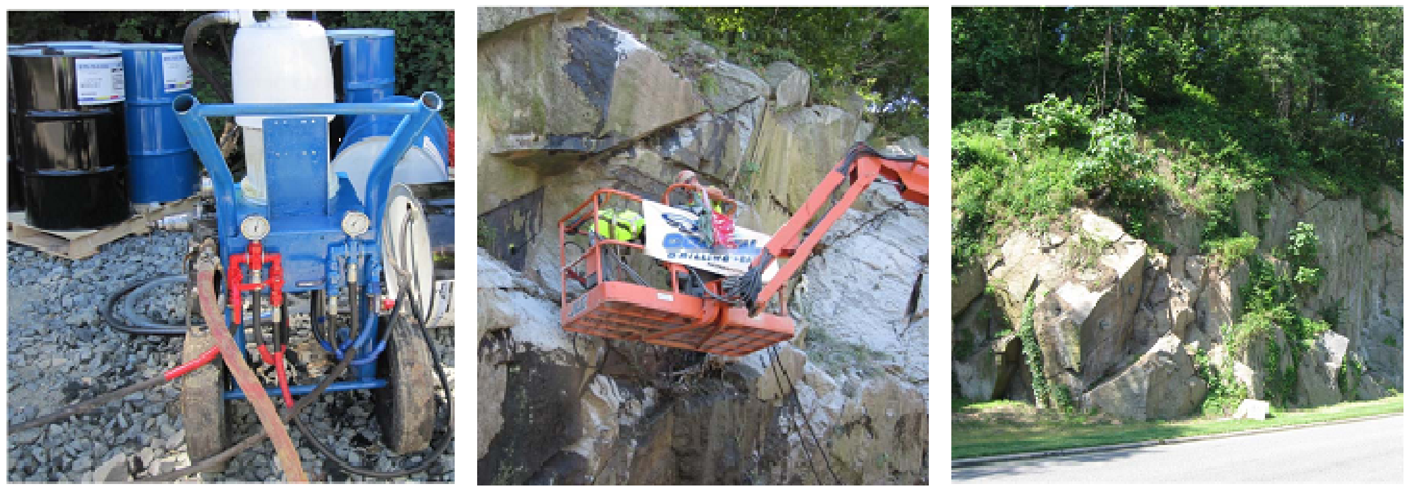
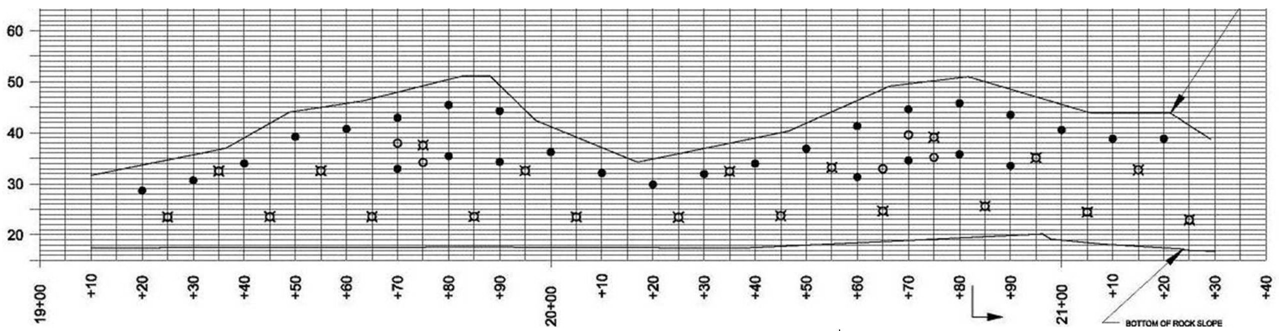


Figure 3: Rock slope consolidated using polyurethane grout to minimize visual impact (source: www.marshall.edu)



Grout pump and drums | Grout injection in predrilled holes | After stabilization



Layout of primary grout holes (●), control holes (○) and drain holes (☼); offsets and elevations in feet

MITIGATION THROUGH REDUCTION OF HAZARD			
5 MODIFYING THE MECHANICAL CHARACTERISTICS OF UNSTABLE MASS			
5.6 LOW PRESSURE GROUTING WITH CEMENTITIOUS OR CHEMICAL BINDER			
APPLICABILITY			
Class	**Descriptor**	**Rating**	**Notes**
Type of movement (Cruden & Varnes, 1996)	Falls	6	General consolidation of rock mass and granular soils. Can treat selected horizons, even at significant depth, making it attractive for spreads
	Topples	4	
	Slides	6	
	Spreads	6	
	Flows	4	
Material	Earth	6	Treatment limited to sand and coarser material
	Debris	8	
	Rock	6	
Depth of movement	Superficial (< 0.5 m)	0	Most efficient when treating medium to deep soils.
	Shallow (0.5 to 3 m)	4	
	Medium (3 to 8 m)	6	
	Deep (8 to 15 m)	8	
	Very deep (> 15 m)	8	
Rate of movement (Varnes, 1978)	Moderately to fast	0	Treatment presupposes that the slide is stable or moving at most very slowly
	Slow	0	
	Very slow	2	
	Extremely slow	8	
Groundwater	Artesian	0	All conditions leading artesian conditions
	High	6	
	Low	8	
	Absent	8	
Surface water	Rain	8	Water courses must be diverted from treatment area. Attention is necessary in very open debris and karsic rock to avoid outflow of grout to water courses
	Snowmelt	8	
	Localized	6	
	Stream	0	
	Torrent	0	
	River	0	
Maturity		6	Limited experience of application to slope stabilization onshore. More widely used for preventive stabilization of marine slopes
Reliability		6	Well developed technology. Difficult to predict outcome. Requires expert supervision and adaptation of design to progress of installation
Implementation		6	Requires specialist equipment and know-how. Relatively small drilling equipment.
Typical Cost		6	Moderate to high, depending on whether cement or chemical grouts are required

Note
Ratings are given on a scale of 1 to 10; the higher the grade, the most suitable is the specific method under consideration to use in landslides of the given characteristics, evaluated individually. Overall suitability to specific case under consideration may be obtained by a weighted average of these ratings, with user defined weights. Zero rating means "not applicable".

MITIGATION THROUGH REDUCTION OF HAZARD

5 MODIFYING THE MECHANICAL CHARACTERISTICS OF UNSTABLE MASS

5. 6 LOW PRESSURE GROUTING WITH CEMENTITIOUS OR CHEMICAL BINDER

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MITIGATION THROUGH REDUCTION OF HAZARD

5 MODIFYING THE MECHANICAL CHARACTERISTICS OF UNSTABLE MASS

5.7 JET GROUTING

Description

Jet-grouting is different from other grouting and deep mixing methods as it erodes and loosens the soils with high pressures and completely mixes the soil with cementitious slurry while gradually withdrawing the injection pipe (Mc Carthy, 2007). The resulting material is often referred to as *soilcrete*, especially when jet grouting is carried out in coarse grained soils.
Jet grouting is carried out as follows:

1. An injection pipe is pushed or drilled into the ground to the desired depth.
2. Grout is injected laterally at high speed from a nozzle located near the end of the pipe into the soil while the pipe is continuously rotated and gradually withdrawn, either continuously or, preferably, in small discrete steps. The procedure is carried on until the whole unstable layers are covered. Three basic systems may be adopted (Figure 1.): single (grout), double (grout and air) and triple fluid (grout, air and water).
3. The procedure is repeated at several locations at a predetermined spacing, usually in a close grid pattern; secant inclusions may be used to form nominally continuous panels where required for stability (see fact sheet 5.0) or for groundwater exclusion.

The addition of air in double and triple fluid systems isolates the eroding jet (grout or water respectively) from the surrounding soil, to achieve greater depths of erosion and thus larger inclusions. Triple jet systems minimize the amount of grout used for erosion.
Jet-grouting may replace a large amount of soil mass; the columns diameter depends on the soil to be treated and on the system used (mono, double or triple fluid); it is typically 0.4 to 2 m for fine grained soils and 0.5 to 3 m for coarse grained soils (Nikbakhtan et al., 2010). Optimization of the nozzle geometry and the use of very high pressure pumps allows the formation of very large inclusions, up to 3 to 5 m wide in the most favourable conditions (Figure 2, Shibazaki, 2003, Mc Carthy, 2007).
In order to achieve the high jet speeds necessary to erode the surrounding soils, the eroding fluid is injected at very high pressure. The pressure is converted into speed at the nozzle and does not materialize in the soil-fluid mix nor in the surrounding soil, provided that a clear outlet is maintained at all times allowing excess fluid and spoil to flow to the surface under low pressure gradients. Severe heaving and/or lateral displacements may occur if this flow is interrupted. To minimize this risk, a cased hole is used in soils where the probehole is prone to instability. The casing is withdrawn simultaneously with the drill string.
Jet grouting inevitably generates large amounts of spoil; in normal conditions the volume of spoil is roughly equivalent to the volume of the inclusion formed. The spoil is a thick soil/grout slurry, not suitable for dry handling (Figure 3).
Jet-grouting is applicable for the whole range of soils and may be applied to any depth down to 50 m (Mc Carthy, 2007); it can be ended at any depth, making it possible to treat only the unstable zone (Jaritngam, 2003).
Very stiff cohesive soils of high plasticity and boulders pose special problems and may limit the applicability of the technique. Active movement may be accelerated by the jet grouting treatment works.

Design

Jet grouted columns act as reinforcement having much better mechanical characteristics than the surrounding soil.
Unless mass treatment is carried out, which is highly unusual, the verification of effectivness of the treatment is complex, since it refers to the behaviour of a discontinuous mass. It can only be addressed by applying significant simplifications.
Available simplified methods are based on limit equilibrium (in static and seismic conditions).
The properties of the inclusions are pre-determined from laboratory tests carried out at different confining pressures to determine the strength envelope of the treated soil in terms of both total and effective stress. Bearing in mind that due to inmperfect mixing filed strengths are typically only 35 to 50% of the strength measured in laboratory tests, the actual strength of the treated soil needs to be verified by trial fields and control tests.
Where the surrounding soil is clay, it can be modelled in terms of undrained shear strength, with appropriate reductions in case of cyclic loads (see for example Idriss and Boulanger, 2008).
Where the surrounding soil is sand, it can be modelled in terms of the least of its drained and its undrained strength; the latter may be evaluated on the basis of empirical correlations as proposed, for example, by Olson and Stark (2002), Olson and Stark (2003a), Olson and Stark (2003b) and Mesri (2007) at pre-liquefaction and post-liquefaction conditions.

Figure 1 Left: Principles of jet grouting Right: The three basic systems of jet grouting (source: Nikbakhtan et al., 2010)

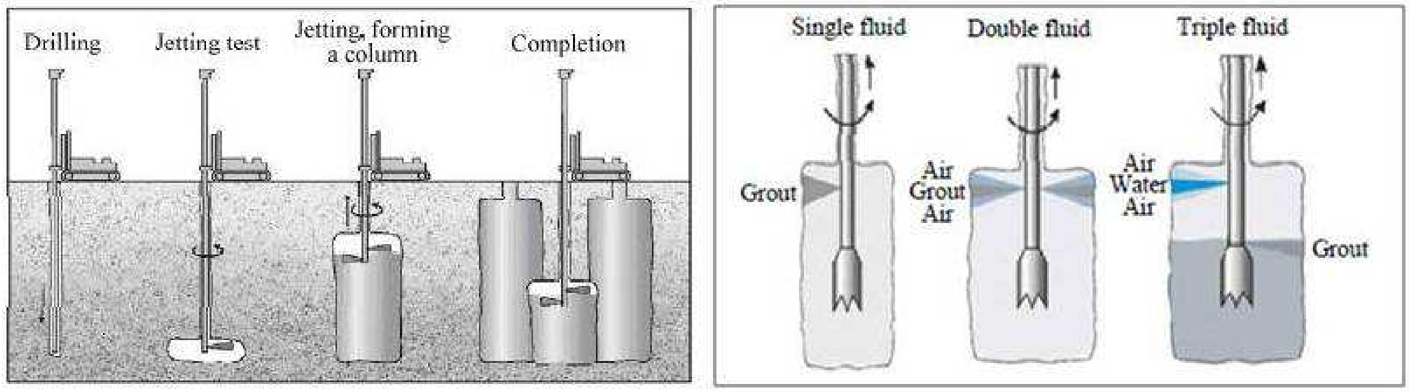

Figure 2: Large soilcrete columns can be formed in favourable conditions (source: Shibazaki, 2003)

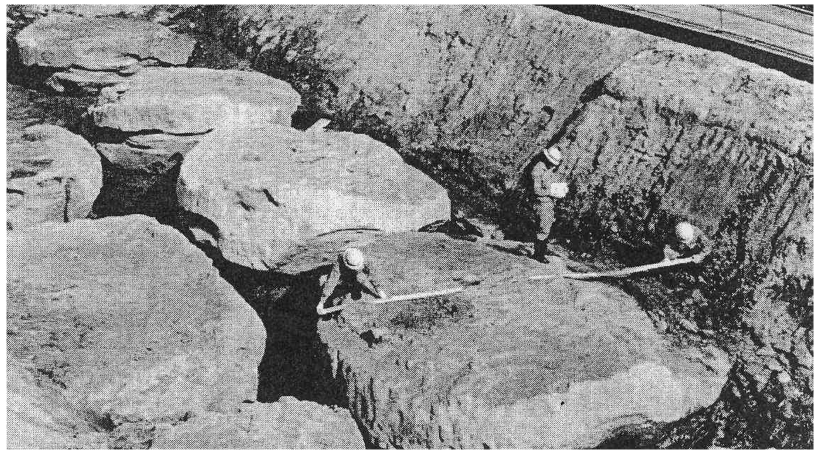

Figure 3: Large amounts of spoil are generated, roughly equivalent to the treated volume (photo: G. Vaciago, SGI-MI)

MITIGATION THROUGH REDUCTION OF HAZARD

5 MODIFYING THE MECHANICAL CHARACTERISTICS OF UNSTABLE MASS

5.7 JET GROUTING

APPLICABILITY

Class	Descriptor	Rating	Notes
Type of movement (Cruden & Varnes, 1996)	Falls	0	Application to landslide stabilization generally limited by need to use relatively heavy equipment. Applicability to spreads and flows to be carefully evaluated on a case by case basis, bearing in mind the risk that installation iteself could trigger movement
	Topples	0	
	Slides	6	
	Spreads	4	
	Flows	4	
Material	Earth	6	Most suited to coarse grained soils. Stiff plastic clay and boulders pose special problems
	Debris	8	
	Rock	0	
Depth of movement	Superficial (< 0.5 m)	0	Typically inappropriate in shallow applications. Selective treatment may be carried out, which makes it potentially suitable for deep lansdslides.
	Shallow (0.5 to 3 m)	0	
	Medium (3 to 8 m)	6	
	Deep (8 to 15 m)	8	
	Very deep (> 15 m)	8	
Rate of movement (Varnes, 1978)	Moderately to fast	0	Workers' safety and end result require construction to take place when movement is extremely slow or very slow (maximum 1.5 m/year or 5 mm/day). Under special conditions and taking due precautions, it may be carried out when movement is "slow" (up to 1.5 m/month, corresponding to 5 cm/day) .
	Slow	2	
	Very slow	6	
	Extremely slow	8	
Groundwater	Artesian	6	Generally applicable in all groundwater conditions. Severe artesian groundwater conditions or strong underground flows may cause seepage induced leaching of the inclusion before the binder sets.
	High	8	
	Low	8	
	Absent	8	
Surface water	Rain	8	Water courses need to be temporarily diverted or reliably dry during construction. Potential pollution of watercourses during construction (for example by spillage of grout or spoil) may impose restriction on construction procedure. No problems once the works are completed, except possibly when treated columns provide an undesired "hard bank" to watercourses.
	Snowmelt	8	
	Localized	8	
	Stream	2	
	Torrent	2	
	River	2	
Maturity		6	The technique is well established, but with limited previous application to the mitigation of natural landslides.
Reliability		6	Geometry and mechanical characteristics of inclusion uncertain, especially in landslides where mixed and variable soil profiles are encountered..
Implementation		5	Requires specialist equipment and techniques; may need temporary roads and working platform for safe operation. Generates significant amounts of spoil
Typical Cost		4	Relatively expensive.

Note
Ratings are given on a scale of 1 to 10; the higher the grade, the most suitable is the specific method under consideration to use in landslides of the given characteristics, evaluated individually. Overall suitability to specific case under consideration may be obtained by a weighted average of these ratings, with user defined weights. Zero rating means "not applicable"

MITIGATION THROUGH REDUCTION OF HAZARD

5 MODIFYING THE MECHANICAL CHARACTERISTICS OF UNSTABLE MASS

5.7 JET GROUTING

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MITIGATION THROUGH REDUCTION OF HAZARD

5 MODIFYING THE MECHANICAL CHARACTERISTICS OF UNSTABLE MASS

5.8 MODIFICATION OF GROUNDWATER CHEMISTRY (E.G. LIME PILES)

Description

The influence of changes in pore water chemistry on the residual strength of clays has been widely reported in the technical literature. Ramiah (1970) reported variations in the residual angle of friction φ'_r of about 4°. Similar conclusions were reached by Kenney (1977), Moore (1991), Di Maio (1996a, 1996b), Maggiò et al. (2002) for various pure and natural clays and by Steward and Cripps (1983) for pyritic shale.

Moore (1991) carried out a systematic laboratory investigation of this issue and reported that clays saturated with monovalent sodium cations consistently resulted in lower residual strengths than clays saturated with calcium divalent cations. The type of cation can account for changes in residual strength of up to approximately 40% for montmorillonite and 15% for kaolinite clay minerals.

The concentration of salts in in the pore water was found to result in furhter differences in residual strength. Moore (1991) also showed that increasing concentrations of seawater result in increasing residual strength in natural clays too, suggesting that seasonal fluctuations in the concentration of salts in pore water can modify the residual strength of natural calys. This observation, which can be particularly significant for coastal landslides, is corroborated by field observations (Moore, 1988; Moore and Brunsden, 1996).

Mesri and Olson (1971) showed that the void ratio of clay samples decreased when subjected to a long term increase in the concentration of NaCl, thus increasing consolidation and stability. NaCl are especially known for long term stability of sensitive clays as the presence of cations change the surface tension on the clay minerals. Long term leaching of NaCl destabilizes clays and when the content of NaCl becomes too low the clay becomes *quick* (NGU, 2002). In spite of this it is not found that NaCl is used for increasing stability of clays by groundwater exchange.

Instead, the most common technique for lowering landslide susceptibility by modification of groundwater chemistry is to add lime to the soil, often creating lime columns in the ground. The methods for creating lime columns in the ground are the same as described for mechanical deep mixing, permeation grouting and jet grouting.

Lime-stabilization has been applied especially to soft and sensitive clays (Rogers and Glendinning, 1997).

It is widely reported that lime migrates from the columns, stabilizing also the surrounding clay. Stabilization is achieved due to formation of calcium silicate hydrate and calcium aluminate hydrate; both gels crystallize in the pores of the clay (Rogers and Glendinning, 1996). The migration has been reported over great distances, probably due to hydraulic gradients. Bell (1996) investigated the effect of lime stabilization on both clay and till and found that till did not show any significant increase in stability to tratment with lime. Migration of ground water into lime columns has also been observed.

The effects of lime columns in clay may be summarized as follows (Rogers and Glendinning, 1997):

- Increased strength of an annular zone of clay surrounding the columns, caused by lime-clay reaction;
- Clay dehydration;
- Generation of negative pore-water pressure;
- Over-consolidation of the soil in the shear plane;
- Columns strength.

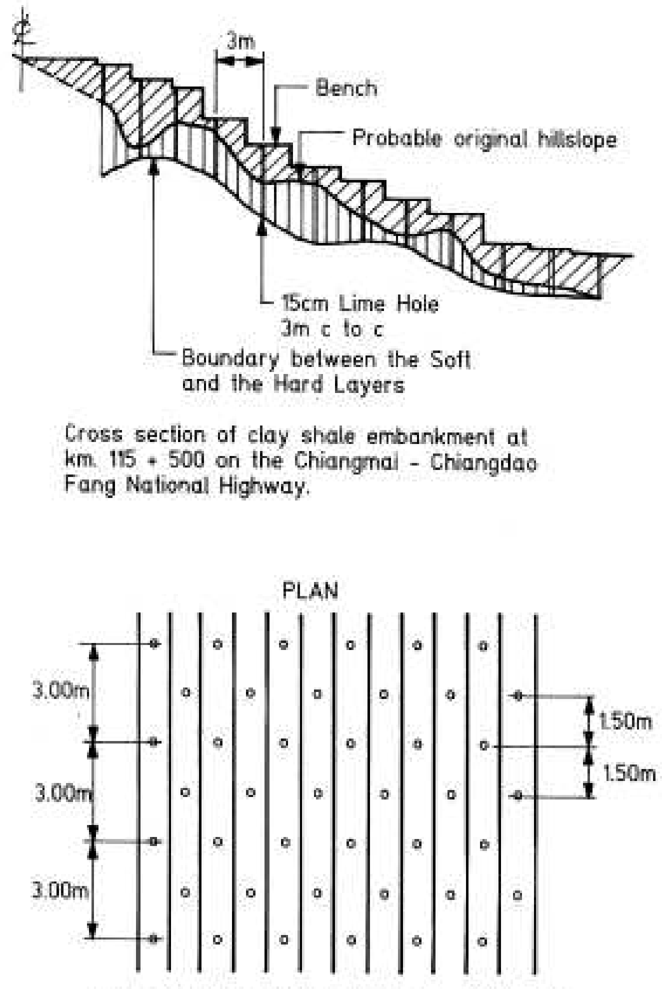
The stabilization of an embankment of loose clay shale fill was attempted in Thailand in 1977. Lime piles were installed in a regular grid with a spacing of 3 m (Figure 1). Holes 15 cm in diameter were augered by hand down to natural hard ground, and lime and water were poured into the holes and topped up daily for two months.

Based on measurements at four locations, Ruenkrairergsa and Pimsarn (1982) report a significant change in soil properties two years after installing the lime piles: the water content of the clay decreased by up to 6.0 %, the cohesion increased by up to 15.7 kN/m² and the friction angle increased by up to 8.1°.

Design

Although some experimental case histories are reported in the literature, some of which characterized by a reasonable degree of success, there is no consolidated and reliable design approach at this stage for landslide stabilization besed on modifications of groundwater chemistry, which at present remains wholly empirical.

Figure 1: : Lime pile stabilization of clay shale embankment – section and plan (source: Ruenkrairergsa and Pimsarn, 1982)



MITIGATION THROUGH REDUCTION OF HAZARD			
5 MODIFYING THE MECHANICAL CHARACTERISTICS OF UNSTABLE MASS			
5.8 MODIFICATION OF GROUNDWATER CHEMISTRY (E.G. LIME PILES)			
APPLICABILITY			
Class	**Descriptor**	**Rating**	**Notes**
Type of movement (Cruden & Varnes, 1996)	Falls	0	Applicability to spreads and flows to be carefully evaluated on a case by case basis, bearing in mind the risk that installation iteself could trigger movement
	Topples	0	
	Slides	6	
	Spreads	4	
	Flows	4	
Material	Earth	6	Only applicable in clays, but stabilizing effects depend on continued treatment
	Debris	0	
	Rock	0	
Depth of movement	Superficial (< 0.5 m)	0	Groundwater chemistry conditioned through relatively small boreholes, can be used in medium to very deep landslides.
	Shallow (0.5 to 3 m)	4	
	Medium (3 to 8 m)	8	
	Deep (8 to 15 m)	8	
	Very deep (> 15 m)	8	
Rate of movement (Varnes, 1978)	Moderately to fast	0	Long term operation of injection boreholes make this technique applicable only when movement is extremely slow or very slow (maximum 1.5 m/year or 5 mm/day)
	Slow	0	
	Very slow	6	
	Extremely slow	8	
Groundwater	Artesian	0	Uses groundwater for diffusion from injection hole to soil mass; best suited to sites with high groundwater levels and a moderate groundwater flow
	High	8	
	Low	4	
	Absent	0	
Surface water	Rain	6	Not applicable close to water courses. Potential pollution of watercourses during construction or from subsequent diffusion of salts..
	Snowmelt	6	
	Localized	0	
	Stream	0	
	Torrent	0	
	River	0	
Maturity		4	Mostly experiemntal at this stage. Some succesful case histories exist, but no established design practice
Reliability		4	Case histories indicate contrasting results. Extend and effectiveness of diffusion unpredicatble. Needs continuous maintenance to remain effective.
Implementation		8	Relatively simple to implement
Typical Cost		6	Installation cost is moderate, but requires maintenance

Note
Ratings are given on a scale of 1 to 10; the higher the grade, the most suitable is the specific method under consideration to use in landslides of the given characteristics, evaluated individually. Overall suitability to specific case under consideration may be obtained by a weighted average of these ratings, with user defined weights. Zero rating means "not applicable"

MITIGATION THROUGH REDUCTION OF HAZARD

5 MODIFYING THE MECHANICAL CHARACTERISTICS OF UNSTABLE MASS

5.8 MODIFICATION OF GROUNDWATER CHEMISTRY (E.G. LIME PILES)

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MITIGATION THROUGH REDUCTION OF HAZARD

FACT SHEET 6

TRANSFER OF LOADS TO MORE COMPETENT STRATA

MITIGATION THROUGH REDUCTION OF HAZARD

6 TRANSFER OF LOADS TO MORE COMPETENT STRATA

6.0 GENERAL

Basic principles and physical process

Mitigation measures in this cathegory operate as a surrogate increase in the resistance of the actual or potential sliding mass either by partially replacing the shear surface with more competent materials (e.g. shear keys, piles, etc. – Figure 1) or by mechanically increasing the effective normal stress on the actual or potential failure surface, thus increasing the shear resistance of the soil or rock (eg. pretensione strand anchors – Figure 2). Some systems operate on both principles simultaneously (eg. passive anchors, soil/rock nailing – Figure 3). In both cases, these measures operate by transferring part of the driving forces to the more competent, stable strata underlying the (actual or potential) sliding mass.

These systems progressively loose their effectiveness as the sliding mass becomes a flowing mass, either through internal processes (eg. loss of microstructure, especially in saturated materials), or through mixing with addition of water from surface runoff or graoundwater.

Design

Refer to the description of the various techniques.

Figure 2

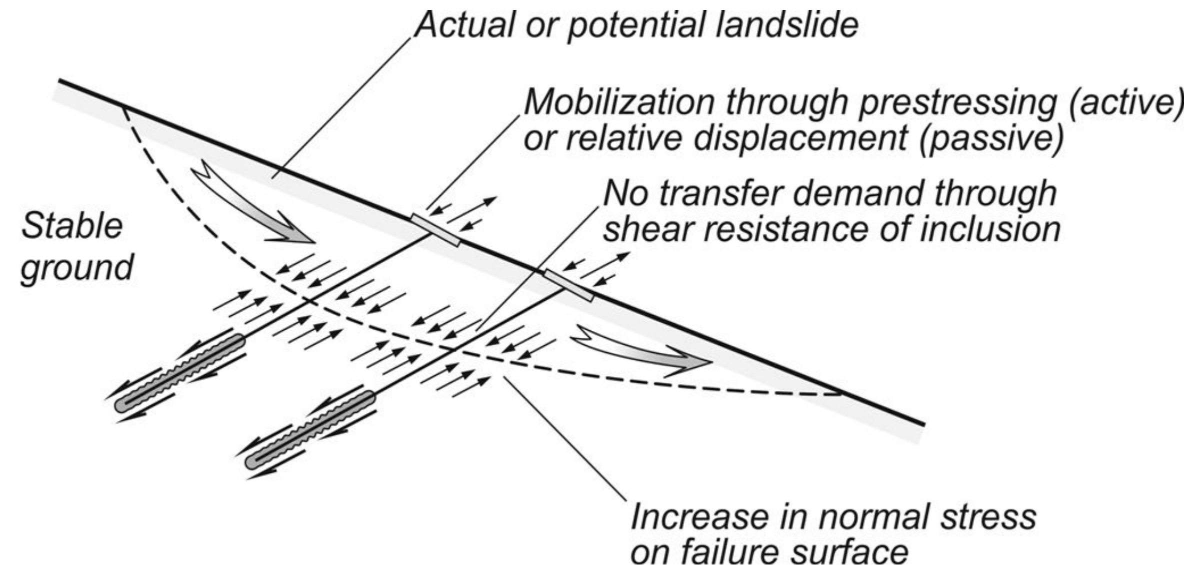


Figure 1

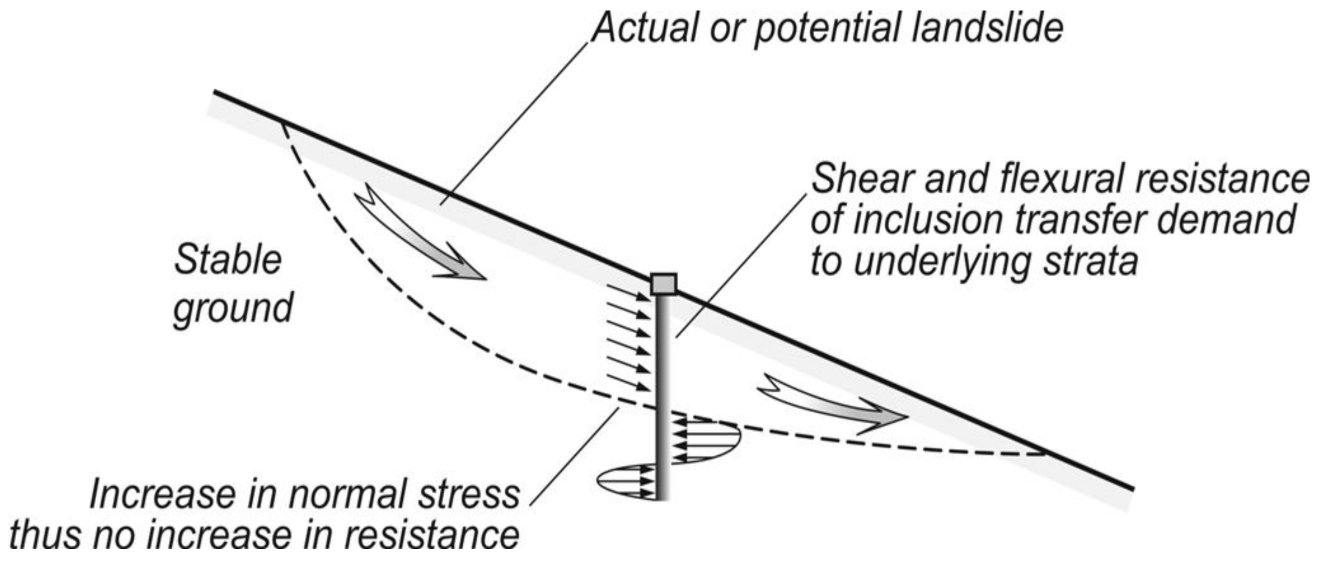
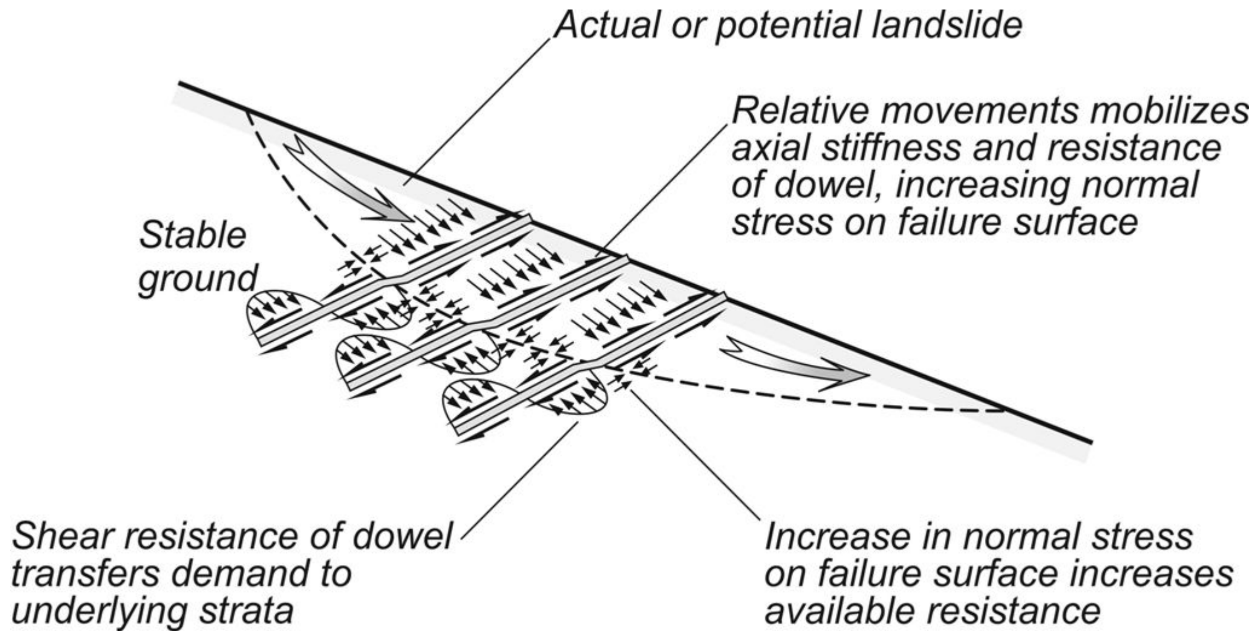


Figure 3



MITIGATION THROUGH REDUCTION OF HAZARD

6 TRANSFER OF LOADS TO MORE COMPETENT STRATA

6.1 COUNTERFORT DRAINS (TRENCH DRAINS INTERSECTING BASAL SHEAR PLANE)

Description

Deep trench drains that intercept the slip plane and provide additional frictional resistance are generally called counterfort drains. Trench drains are commonly used to stabilize landslides of small to moderate depth in clay slopes. They contribute to slope stability only through their drainage action, as discussed in detail in the relevant fact sheets. If trench drains are deep enough to intersect the basal failure plane, they provide additional mechanical stabilization, by the replacement of the weak slipped material by the stronger material in the drain, thus improving the average shear resistance that can be mobilized on the failure plane for any given pore pressure regime (Lee and Clark, 2002). While deep trench drains intersecting the failure plane are generally referred to as "counterfort drains", the term is often used loosely to indicate trench drains aligned along or close to the direction of maximum inclination of the slope, irrespective of whether they do or do not intersect the slip plane.

One of the earliest formally reported applications of counterfort drains to stabilize landslide is the construction of deep gravel filled counterfort drains through the shear surface to the undisturbed clay below to remediate rotational movements observed in London Clay in railway cuttings at New Cross (Gregory, 1844).

Deep counterfort drains are reported by Tianchi (1996) to be the main measure used to treat small and medium scale landslides because of the combined benefits of the drainage and mechanical effects.

Many slip planes are less than 5 m deep and counterfort drains can be excavated to 6 m deep using hydraulic backactor excavators; greater depths up to 7 or 8 m deep can be reached using machines equipped with long reach booms. They are typically 0.5 to 1.0 m wide and they are back-filled with suitable free-draining material. They are design as invertes filters, with a gravel core surrounded by sand, to prevent them becoming chocked with fines, which renders them ineffective. Geotextile filters are widely used for this purpose to simplify construction. A porous pipe may be placed at the base to collect and remove the water. Provision to prevent clogging must be incorporated in the design. The mechanical benefits are increased if free draining concrete is used in lieu of the gravel fill.

Design

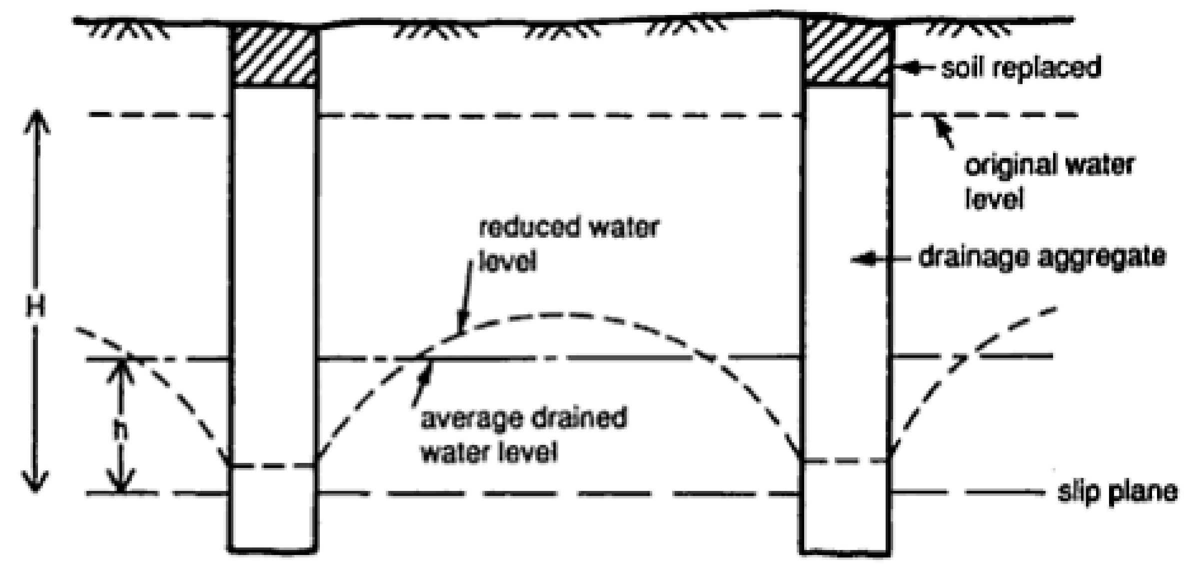
For the hydraulic aspect of the design, reference shall be made to the relevant fact-sheets.

Provided the length, thickness and spacing of the counterfort drains are such that load transfer from the sliding mass to the counterforts and from the these to the underlying stable soil is guaranteed, the mechanical benefit of partially replacing the shear surface with more competent material may be taken into account simply by calculating the post construction average strength as the weighted average strength of the original soil and the drain material.

Clearly, this is most effective when remediating pre-existing planar slides in clay, which often exist close to limit equilibrium and are cyclically reactivated. Assuming a residual angle of friction on the failure plane equal to 14° and an angle of friction of the drain material equal to 32°, a replacement ratio of 20% would result in a 30% improvement in the factor of safety of the slope. Clearly, lower replacement ratios are sufficient to provide a similar result if the drainage effect is also taken into account.

For the full mechanical effect to be mobilized, the proportions between the length and the spacing of the drains must be such that arching takes place between adjacent drains.

Figure 1: Schematic section. Counterfort drains provide additional stabilization by intercepting the slip plane (source: Carter, 1992)



Picture 1: Deep trench drains (counterfort drains) under construction (source: http://www.svr-vlo.org.uk/floodline_no50-59.htm)

MITIGATION THROUGH REDUCTION OF HAZARD

6 TRANSFER OF LOADS TO MORE COMPETENT STRATA

6.1 COUNTERFORT DRAINS (TRENCH DRAINS INTERSECTING BASAL SHEAR PLANE)

APPLICABILITY

Class	Descriptor	Rating	Notes
Type of movement (Cruden & Varnes, 1996)	Falls	0	Applicable to planar slides and, to a lesser extent, to rotational slides.
	Topples	0	
	Slides	8	
	Spreads	0	
	Flows	0	
Material	Earth	8	Most suitable in clays, both in terms of ease and local stability of excavations and in terms of relative effectivness. In debris it may be useful if carried out with free draining concrete.
	Debris	4	
	Rock	0	
Depth of movement	Superficial (< 0.5 m)	8	Depths up to 4 – 5 m can be reached without special difficulty; higher depths up to 7 to 8 m, suitable for slides up to 6 m deep, may be achieved using special equipment (long reach booms).
	Shallow (0.5 to 3 m)	8	
	Medium (3 to 8 m)	4	
	Deep (8 to 15 m)	0	
	Very deep (> 15 m)	0	
Rate of movement (Varnes, 1978)	Moderately to fast	0	Should be carried out preferably on very or extremely slow landslides; with due care it can be carried out in slow landslide.
	Slow	4	
	Very slow	8	
	Extremely slow	8	
Groundwater	Artesian	4	High groundwater levels imply the maximum effectiveness in terms of drainage, but may pose problems during construction; applicability to situations with arrtesian conditions to be reviewed carefully.
	High	8	
	Low	6	
	Absent	2	
Surface water	Rain	6	Suitable to deal with diffused surface water. Concentrated flows should be prevented or diverted from the slope.
	Snowmelt	6	
	Localized	4	
	Stream	2	
	Torrent	0	
	River	0	
Maturity		8	Traditional technique, widely applied, mainly on an empirical basis without formal design.
Reliability		8	Generally reliable. Exact location of slip surface can be confirmed by inspection during installation. Effective almost immediately.
Implementation		6	Deep excavation in potentially unstable soil causes significant safety hazard. must be well planned. Arrangements must be made to avoid man entry.
Typical Cost		8	Relatively low cost, unless free draining concrete is used and provided suitable material is readily available.

Note
Ratings are given on a scale of 1 to 10; the higher the grade, the most suitable is the specific method under consideration to use in landslides of the given characteristics, evaluated individually. Overall suitability to specific case under consideration may be obtained by a weighted average of these ratings, with user defined weights. Zero rating means "not applicable"

MITIGATION THROUGH REDUCTION OF HAZARD	
6 TRANSFER OF LOADS TO MORE COMPETENT STRATA	
6.1 COUNTERFORT DRAINS (TRENCH DRAINS INTERSECTING BASAL SHEAR PLANE)	
References: Carter P. (1992) "Dewatering". In: Drainage Design, P. Smart and J.G. Herbertson editors, blackie & Son Ltd, Glasgow Gregory C. H. (1844) "On railway cuttings and embankments with an account of some slips in London Clay". Minutes of Proceedings of the Institution of Civil Engineers, 3, 135-173 Lee E.M., Clark A.R. (2002) "Investigation and management of soft rock cliffs". Thomas Telford, London, UK Tianchi L. (1996). "Landslide Hazard mapping and management in China". International Centre for Integrated Mountain Development, Kathmandu, Nepal.	

MITIGATION THROUGH REDUCTION OF HAZARD

6 TRANSFER OF LOADS TO MORE COMPETENT STRATA

6.2 PILES

Description

Piles can be placed in earth and debris slopes, either at regular 2D spacing over the whole slide or portion thereof, to act as isolated dowels, or, more commonly, at close spacing along one or more specific alignments to form piled walls across the direction of movement (Ito et al., 1982; Hassiotis and Chameau, 1984; Soric and Kleiner, 1986; Popescu, 1991; Reese et al., 1992; Polysou et al., 1998; Poulos, 1999) - Figure 1.

Typically, large diameter bored cast-in-situ piles are used, with diameter 800 to 2000 (most frequently 1200) mm and spacing 1.2 to 2 times the pile diameter. The advantages of this technique may be summarized as follows:

- applicable in a variety of topographical conditions, subject to access constraints;
- casings limit hole instability during construction and damage to green concrete in piles formed in moving slides;
- conventional equipment may overcome thin layers of rock.

Where access is difficult and/or the depth of sliding is modest, micropiles (200 to 300 mm diameter) are also used, normally reinforced by steel pipes to maximize bending and shear resistance of the micropiles.

Pile heads are usually completed by a capping beam to allow:

- redistribution of horizontal loads between piles;
- the installation of anchors, where required to improve the resistance of the wall;
- the installation of sub-horizontal drains, where required to reduce the thrust on the wall.

Examples of applications are provided by Wilson (1970), Palladino and Peck (1972), Nethero (1982), Oackland and Chameau (1984), Isenhower et al. (1989), Rollins and Rollins (1992), Reese et al. (1992), Leoni and Manassero (2003).

Design

The design load on the pile wall may be determined in 2D limit equilibrium analyses by calculating the reaction on the vertical section corresponding to the piled wall which is necessary to guarantee, with the appropriate factor of safety, the stability of the portion of the slide located upslope of the wall in the absence of the downslope portion; in any case, the load on the wall cannot exceed passive soil pressure.

The contribution of the downslope portion can be considered only if this portion remains stable with an appropriate factor of safety once the driving force from the upper portion is removed; even in this case, it may be prudent to consider this mass only as confinement for the stable soil below, since even very small deformation such as shrinkage in a dry season may be sufficient to reduce or completely remove downslope support to the wall.

The design loads and the stability of the downslope portion in seismic conditions are normally determined from pseudostatic limit equilibrium analyses, taking into account the excess pore pressures that may develop in the slope, where applicable.

Once the net actions imposed by the landslide on the pile wall are known, a suitable soil-structure interaction analysis is carried out by an appropriate method to determine both the reactions in the stable soil into which the piles are anchored and the effects of actions on the piles.

The spacing between the piles must be determined balancing:

- economy and the need to avoid interference between adjacent piles during construction and with natural drainage;
- ensuring that soil arching develops between adjacent piles and that the soil does not “flow” between the piles.

The check that soil arching develops between adjacent piles and that the soil does not “flow” through the piles can be done by means of analytical (simplified) tools (see for example Ito and Matsui, 1975) or 3D numerical analysis.

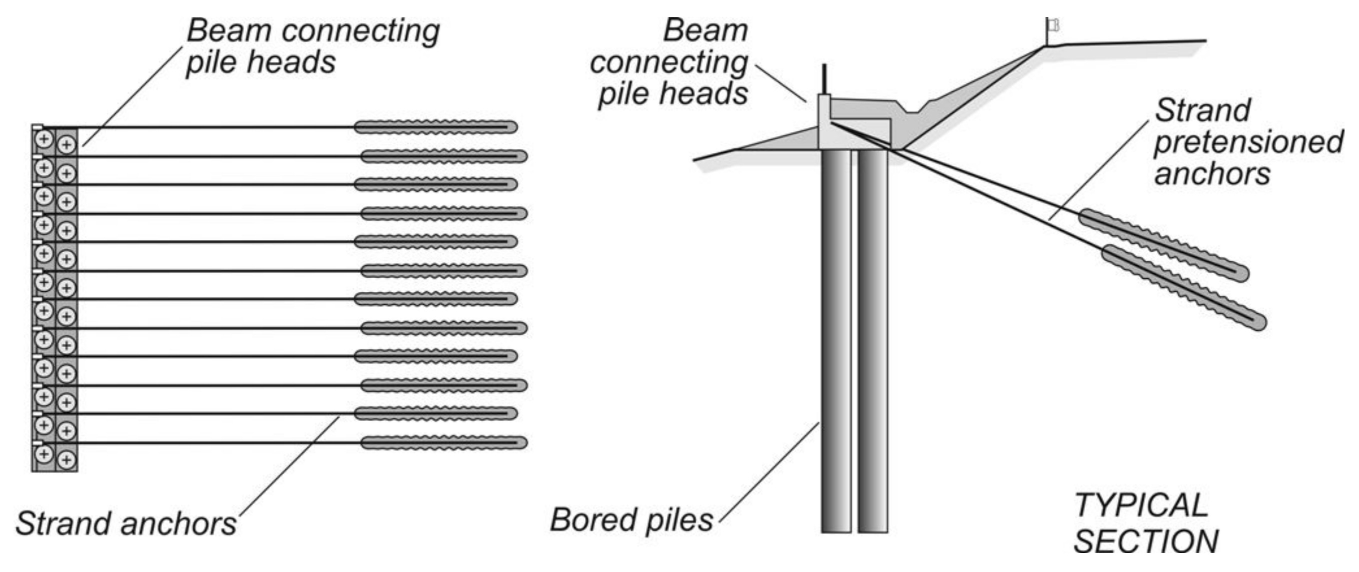
Provided soil arching is guaranteed, plain strain 2D soil-structure interaction analysis is representative of actual conditions, with the effects of actions on each pile being those derived from the 2D analyses, multiplied by the pile centre to centre spacing. The same analysis may be used to determine the optimal length of the piles and the benefit of anchors.

The calculation of the pile capacity in relation to the soil/structure interaction may be carried out according to several approaches and simplified methods (De Beer, 1977; Viggiani, 1981; Hassiotis and Chameau, 1984; Cantoni et al, 1989; Pearlman and Withiam, 1992).

Finite elemnt methods may be used instead to provide a simultaneous and consistent estimate of the soil-structure interaction both with the sliding mass and with the underlying stable soil. Finite element analyses in the time domain can also be used to refine the evaluation of the performance of the structure under seismic conditions.

The mechanical charateristics of the piles must be adequate to sustain the actions and the effects of actions on the piles. The structural checks must satisfy all applicable codes and standards on the subject.

Figure 1: Schematic section and layout (source: SGI-MI project files)



Picture 1: Double row of large diameter piles (source: SGI-MI project files)

MITIGATION THROUGH REDUCTION OF HAZARD

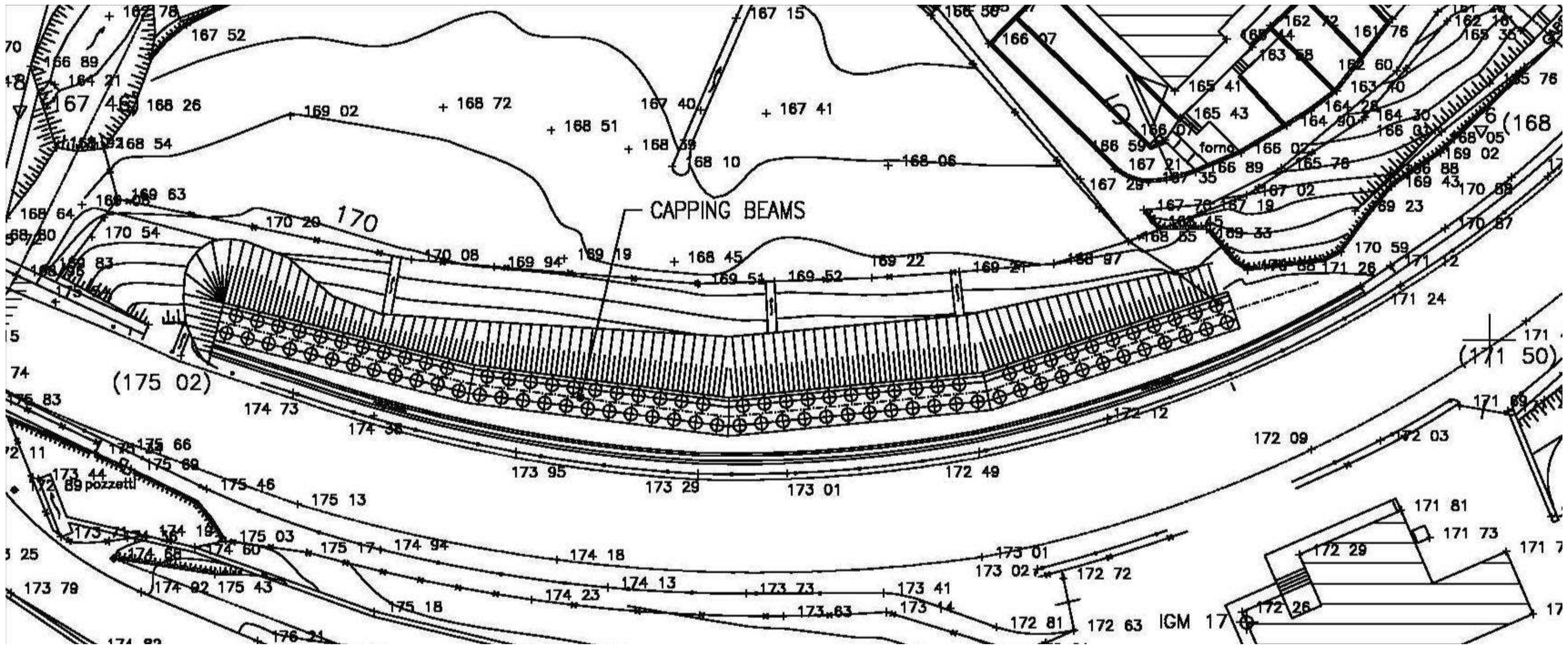
6 TRANSFER OF LOADS TO MORE COMPETENT STRATA

6.2 PILES

Picture 2: Capping beam connecting pile and anchor heads (source: SGI-MI project files)

Picture 3: Rows of micropiles reinforced by steel pipes (source: SGI-MI project files)

Figure 2: Typical layout (source: SGI-MI project files)



MITIGATION THROUGH REDUCTION OF HAZARD

6 TRANSFER OF LOADS TO MORE COMPETENT STRATA

6.2 PILES

APPLICABILITY

Class	Descriptor	Rating	Notes
Type of movement (Cruden & Varnes, 1996)	Falls	0	Best suited to slides and the slide-like portion of complex landslides. May be applicable in some cases to prevent the triggering of slides with the potential to turn to spreads or flows, but are substantially ineffective once fuidification has occurred.
	Topples	0	
	Slides	8	
	Spreads	4	
	Flows	4	
Material	Earth	8	Difficult, very expensive and typically inappropriate in rock. Tools and temporary hole support to be selected taking into account ground conditions. Special care must be excercized where the ground contains large boulders which preferably should be overcome without causing excessive vibration.
	Debris	8	
	Rock	0	
Depth of movement	Superficial (< 0.5 m)	0	Typically: • best suited where the movement is medium deep (3 to 8 m), • inappropriate in shallower movements because excessive, • difficult (large diameter, multiple rows) in deep movements, • not applicable in very deep movements.
	Shallow (0.5 to 3 m)	4	
	Medium (3 to 8 m)	8	
	Deep (8 to 15 m)	4	
	Very deep (> 15 m)	0	
Rate of movement (Varnes, 1978)	Moderately to fast	0	Workers' safety and end result require construction to take place when movement is extremely slow or very slow (maximum 1.5 m/year, corresponding to approximately 5 mm/day). Under special conditions and taking due precautions (permanent casing; drilling non-stop to avoid blokage and brocken piles, it may be carried out when movement is "slow" (up to 1.5 m/month, corresponding to 5 cm/day) .
	Slow	4	
	Very slow	8	
	Extremely slow	8	
Groundwater	Artesian	2	High groundwater levels can be dealt with by standard pile construction procedures, bu artesian groundwater conditions pose special problems during construction, possibly making piles not feasible in extreme cases.
	High	6	
	Low	8	
	Absent	8	
Surface water	Rain	8	Water courses need to be temporarily diverted or reliably dry during construction. Potential pollution of watercourses by piling operations (for example by drilling fluid and/or by grout) may impose restriction on construction procedure. No problems once the works are completed, except possibly when piles provide an undesired "hard bank" to watercourses.
	Snowmelt	8	
	Localized	8	
	Stream	2	
	Torrent	2	
	River	2	
Maturity		10	Technique and design process are well established and widely used in suitable conditions.
Reliability		8	Reliable performance in well characterized landslides; in first time slides it depends on estimate of piezometric regime and apprporiate operational strength parameters of soil, which can be problematic; problems may occur during construction, for example if unforeseen boulders are encountered.
Implementation		6	Requires specialist equipment and techniques; implementation may need temporary roads and working platform for safe operation.
Typical Cost		4	Relatively expensive.

Note
Ratings are given on a scale of 1 to 10; the higher the grade, the most suitable is the specific method under consideration to use in landslides of the given characteristics, evaluated individually. Overall suitability to specific case under consideration may be obtained by a weighted average of these ratings, with user defined weights. Zero rating means "not applicable"

MITIGATION THROUGH REDUCTION OF HAZARD

6 TRANSFER OF LOADS TO MORE COMPETENT STRATA

6.2 PILES

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MITIGATION THROUGH REDUCTION OF HAZARD

6 TRANSFER OF LOADS TO MORE COMPETENT STRATA

6.3 BARRETTES (DIAPHRAGM WALLS)

Description

Barretts (diaphragm wall elements) used for mechanical stabilization of landslides are typically 800 to 1200 mm in thickness and 2000 to 3000 mm in length, matching the size of the equipment used (Leoni and Manassero, 2003). If necessary, multiple panels can be excavated and cast jointly, to form special shapes, such as Tee, or to make longer panels typically up to almost three times the standard panel length, although. They can be placed in earth and debris slopes, typically at a maximum centre to centre spacing of twice the thickness, with the longitudinal axis aligned with the direction of movement, to form specific alignements across the direction of movement at strategic positions within the landslide (Ito et al., 1982; Hassiotis and Chameau, 1984; Soric and Kleiner, 1986; Popescu, 1991; Reese et al., 1992; Polysou et al., 1998; Poulos, 1999)

Construction of the barrettes involves three main stages:

4. Formation of guide walls defining the proposed shape and location of the barrette;
5. Excavation, typically by means of rope or kelly operated clam shells grabs or by hydromills, depending on the nature of the ground to be excavated; a suitable drilling fluid, typically bentonitic mud or similar, is used to support the sides of the excavation; the drilling fluid is also essential to transport the cuttings in reverse circulation when using hydromills.
6. Backfilling with reinforced concrete; after cleaning the hole, for example by forced circulation of the drilling fluid with a high pressure, high capacity pump, the reinforcement cage is installed and concreting proceeds from the base upwords using a tremie pipe, to displace the drilling fluid, which is recovered to temporary storage for reuse in the next barrette.

The advantages of this technique may be summarized as follows:

- applicable in a variety of topographical conditions, subject to access constraints;
- applicable in relatively deep landslides (up to 15÷20 m deep) where other techniques may prove inadequate;
- conventional equipment may overcome thin layers of rock; hydromills can be used to cut into rock;

The heads of the barrettes are usually completed by a capping beam to allow:

- redistribution of horizontal loads between barrettes;
- the installation of anchors, where required to improve the overall resistance of the structure;
- the installation of sub-horizontal drains, where required.

Design

The design load on the barrettes may be determined in 2D limit equilibrium analyses by calculating the reaction on the vertical section corresponding to the barrettes which is necessary to guarantee, with the appropriate factor of safety, the stability of the portion of the slide located upslope of the barrettes in the absence of the downslope portion; in any case, the load on the barrettes cannot exceed passive soil pressure

The contribution of the downslope portion can be considered only if this portion remains stable with an appropriate factor of safety once the driving force from the upper portion is removed; even in this case, it may be prudent to consider this mass only as confinement for the stable soil below, since even very small deformation such as shrinkage in a dry season may be sufficient to reduce or completely remove downslope support to the barrettes.

The design loads and the stability of the downslope portion in seismic conditions are normally determined from pseudostatic limit equilibrium analyses, taking into account the excess pore pressures that may develop in the slope, where applicable.

Once the net actions imposed by the landslide on the barrettes are known, a suitable soil-structure interaction analysis is carried out by an appropriate method to determine both the reactions in the stable soil into which the barrettes are anchored and the effects of actions on them.

The spacing between barrettes must be determined balancing:

- economy and the need to avoid interference between adjacent piles during construction and/or with natural drainage;
- the need to ensure that soil arching develops between adjacent barrettes and that the soil does not “flow” between them.

The check that soil arching develops between adjacent barrettes and that the soil does not “flow” through them can be done by means of analytical (simplified) tools (see for example Ito and Matsui, 1975) or 3D numerical analysis.

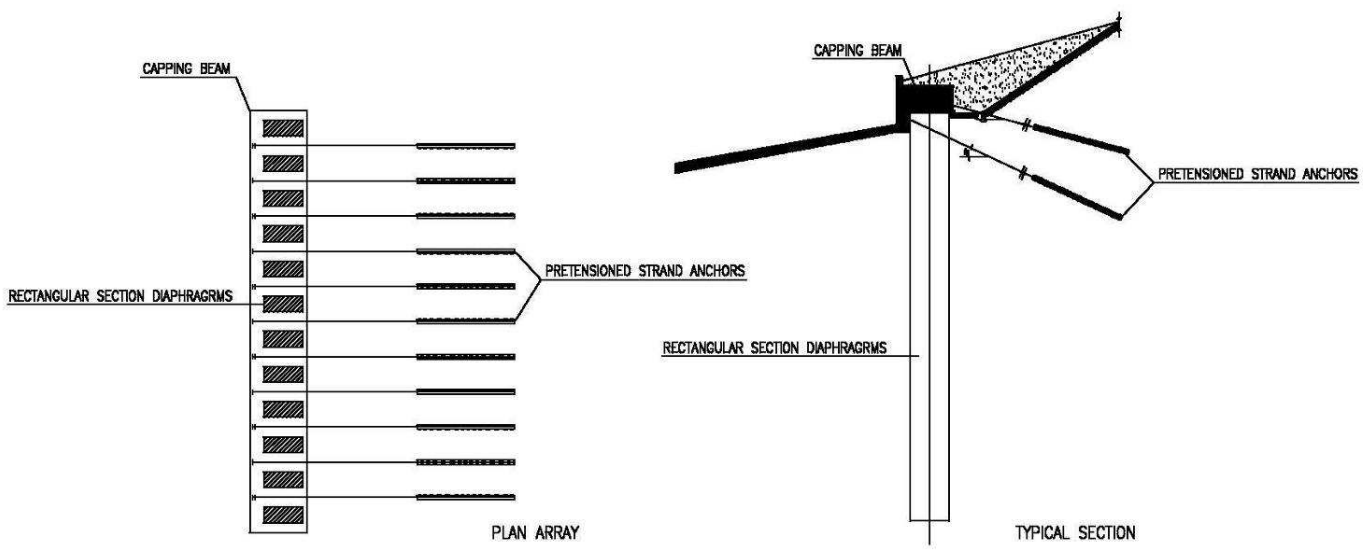
Provided soil arching is guaranteed, plain strain 2D soil-structure interaction analysis is representative of actual conditions, with the effects of actions on each barrette being those derived from the 2D analyses, multiplied by the centre to centre spacing of the barrettes. The same analysis may be used to determine their optimal length and the benefit of additional anchors, if used.

The calculation of the barrettes capacity in relation to the soil/structure interaction may be carried out according to several approaches and simplified methods (De Beer, 1977; Viggiani, 1981; Hassiotis and Chameau, 1984; Cantoni et al, 1989; Pearlman and Withiam, 1992).

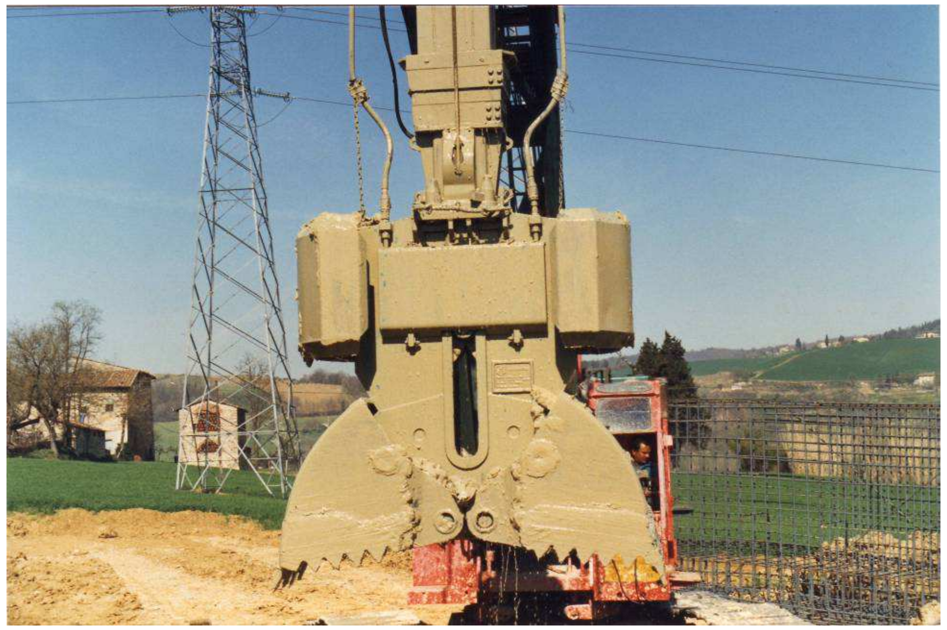
Finite elemnt methods may be used instead to provide a simultaneous and consistent estimate of the soil-structure interaction both with the sliding mass and with the underlying stable soil. Finite element analyses in the time domain can also be used to refine the evaluation of the performance of the structure under seismic conditions.

The mechanical charateristics of the barrettes must be adequate to sustain the actions and the effects of actions on them. The structural checks must satisfy all applicable codes and standards on the subject.

Figure 1: Schematic plan and section (source: SGI-MI project files)



Picture 1: Kelly operated grab for excavation of barrettes and diaphragm walls (source: SGI-MI project files)



MITIGATION THROUGH REDUCTION OF HAZARD

6 TRANSFER OF LOADS TO MORE COMPETENT STRATA

6.3 BARRETTES (DIAPHRAGM WALLS)

Picture 2: Hydromill for excavation of barrettes and diaphragm walls (source: SGI-MI project files)

Picture 3: Excavation in progress; note guide walls and guide frame (source: SGI-MI project files)

Picture 3: Steel reinforcing cage for diaphragm panel - note T-shape (source: SGI-MI project files)

Picture 4: Casting barrette with tremie pipe (source: SGI-MI project files)

MITIGATION THROUGH REDUCTION OF HAZARD

6 TRANSFER OF LOADS TO MORE COMPETENT STRATA

6.3 BARRETTES (DIAPHRAGM WALLS)

APPLICABILITY

Class	Descriptor	Rating	Notes
Type of movement (Cruden & Varnes, 1996)	Falls	0	Best suited to slides and the slide-like portion of complex landslides. May be applicable in some cases to prevent the triggering of slides with the potential to turn to spreads or flows, but are substantially ineffective once fuidification has occurred.
	Topples	0	
	Slides	8	
	Spreads	4	
	Flows	4	
Material	Earth	8	Difficult, very expensive and typically inappropriate in rock. Tools and temporary hole support to be selected taking into account ground conditions. Special care must be excercized where the ground contains large boulders which preferably should be overcome without causing excessive vibration.
	Debris	8	
	Rock	0	
Depth of movement	Superficial (< 0.5 m)	0	Typically: • best suited where the movement is medium deep (3 to 8 m), • inappropriate in shallower movements because excessive, • difficult (large diameter, multiple rows) in deep movements, • not applicable in very deep movements.
	Shallow (0.5 to 3 m)	0	
	Medium (3 to 8 m)	6	
	Deep (8 to 15 m)	8	
	Very deep (> 15 m)	4	
Rate of movement (Varnes, 1978)	Moderately to fast	0	Workers’ safety and end result require construction to take place when movement is extremely slow or very slow (maximum 1.5 m/year, corresponding to approximately 5 mm/day). Under special conditions and taking due precautions (permanent casing; drilling non-stop to avoid blokage and brocken piles, it may be carried out when movement is ”slow” (up to 1.5 m/month, corresponding to 5 cm/day) .
	Slow	2	
	Very slow	6	
	Extremely slow	8	
Groundwater	Artesian	2	High groundwater levels can be dealt with by standard pile construction procedures, bu artesian groundwater conditions pose special problems during construction, possibly making piles not feasible in extreme cases.
	High	6	
	Low	8	
	Absent	8	
Surface water	Rain	8	Water courses need to be temporarily diverted or reliably dry during construction. Potential pollution of watercourses by piling operations (for example by drilling fluid and/or by grout) may impose restriction on construction procedure. No problems once the works are completed, except possibly when piles provide an undesired ”hard bank” to watercourses.
	Snowmelt	8	
	Localized	8	
	Stream	2	
	Torrent	2	
	River	2	
Maturity		10	Technique and design process are well established and widely used in suitable conditions.
Reliability		8	Reliable performance in well characterized landslides; in first time slides it depends on estimate of piezometric regime and apprporiate operational strength parameters of soil, which can be problematic; problems may occur during construction, for example if unforeseen boulders are encountered.
Implementation		6	Requires specialist equipment and techniques; implementation may need temporary roads and working platform for safe operation.
Typical Cost		4	Relatively expensive.

Note
Ratings are given on a scale of 1 to 10; the higher the grade, the most suitable is the specific method under consideration to use in landslides of the given characteristics, evaluated individually. Overall suitability to specific case under consideration may be obtained by a weighted average of these ratings, with user defined weights. Zero rating means ”not applicable”

MITIGATION THROUGH REDUCTION OF HAZARD

6 TRANSFER OF LOADS TO MORE COMPETENT STRATA

6.3 BARRETTES (DIAPHRAGM WALLS)

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MITIGATION THROUGH REDUCTION OF HAZARD

6 TRANSFER OF LOADS TO MORE COMPETENT STRATA

6.4 CAISSONS – MECHANICAL EFFECTS

Description

Caissons used to provide a mechanical stabilization of landslides typically range in diameter between 6 and 15 m (Brandl, 1988; Leoni and Manassero, 2003). They can be placed in earth and debris slopes, typically along specific alignements across the direction of movement at strategic positions within the landslide, at a maximum centre to centre spacing of twice the diameter.

Construction of caissons involves three main stages:

1. Construction of the annular structure which is necessary to ensure that subsequent activities can be carried out safely;
2. Excavation to the design depth, as necessary to ensure that each caisson is adequately keyed into the underlaying competent and stable strata;
3. Backfilling with reinforced concrete (mass concrete may be used in relatively short, large caissons where shear behaviour predominates).

Depending on anticipated ground and groundwater conditions, the most common techniques used to form the annular structure constructed in the first stage are (De Paoli, 1989; Tambara, 1999):

- Progressive construction during excavation by alternate excavation and casting of consecutive concrete rings, although this may be problematic in unstable slopes.
- Advance formation of an annular structure by means of micropiles, jet grouted columns, piles or diaphragm walls, which is later supplemented by annular steel or concrete ribs as excavation proceeds.

Where ground conditions vary significantly along the depth to be excavated, different techniques can be used for different portions of the structure: for example by performing the annular structure to rockhead only and extending the excavation into rock with local support only.

Special care needs to be paid when excavating below the groundwater level, especially if more permeable ground is overlain by less permeable ground and/or where running conditions may occur. Temporary dewatering is necessary in these conditions and in extreme cases they may make this technique inapplicable.

The main advantages of this technique may be summarized as follows:

- Very stiff and robust structure;
- Applicable in deep landslides (up to 20÷25 m deep) where other techniques may prove inadequate;
- Main structural components are constructed under controlled, clean conditions, allowing inspection of reinforcement and controlled placement and compaction of concrete;
- May be adapted to suit a variety of ground conditions below the sliding mass, including rock;
- Allow installation of anchors and/or suborizontal drains from within the caissons, several metres below ground level;
- Allow direct inspection of sliding mass and underlying competent strata during construction.

On the contrary, it must be borne in mind that construction may take several months and it requires access roads and a level working platform for safe operation, which on relatively steep ground may require significant preliminary works.

Design

The design load on the caissons may be determined in 2D limit equilibrium analyses by calculating the reaction on the vertical section corresponding to the caisson alignement which is necessary to guarantee, with the appropriate factor of safety, the stability of the portion of the slide located upslope of the wall in the absence of the downslope portion; in any case, the load on the wall cannot exceed passive soil pressure.

The contribution of the downslope portion can be considered only if this portion remains stable with an appropriate factor of safety once the driving force from the upper portion is removed; even in this case, it may be prudent to consider this mass only as confinement for the stable soil below, since even very small deformation such as shrinkage in a dry season may be sufficient to reduce or completely remove downslope support to the caissons.

The design loads and the stability of the downslope portion in seismic conditions are normally determined from pseudostatic limit equilibrium analyses, taking into account the excess pore pressures that may develop in the slope, where applicable.

Once the net actions imposed by the landslide on the caissons are known, a suitable soil-structure interaction analysis is carried out by an appropriate method to determine both the reactions in the stable soil into which the caissons are anchored and the effects of actions on the caissons.

The spacing between the caissons must be determined balancing:

- economy and the need to avoid interference between adjacent caissons and/or with natural drainage;
- the need to ensure that soil arching develops between adjacent caissons and that the soil does not “flow” between them.

The check that soil arching develops between adjacent caissons and that the soil does not “flow” between them can be done by means of analytical (simplified) tools (see for example Ito and Matsui, 1975) or 3D numerical analysis.

Provided soil arching is guaranteed, plain strain 2D soil-structure interaction analysis is representative of actual conditions, with the effects of actions on each caisson being those derived from the 2D analyses, multiplied by their centre to centre spacing. The same analysis may be used to determine the optimal length of the caissons and the benefit of additional anchors, if used.

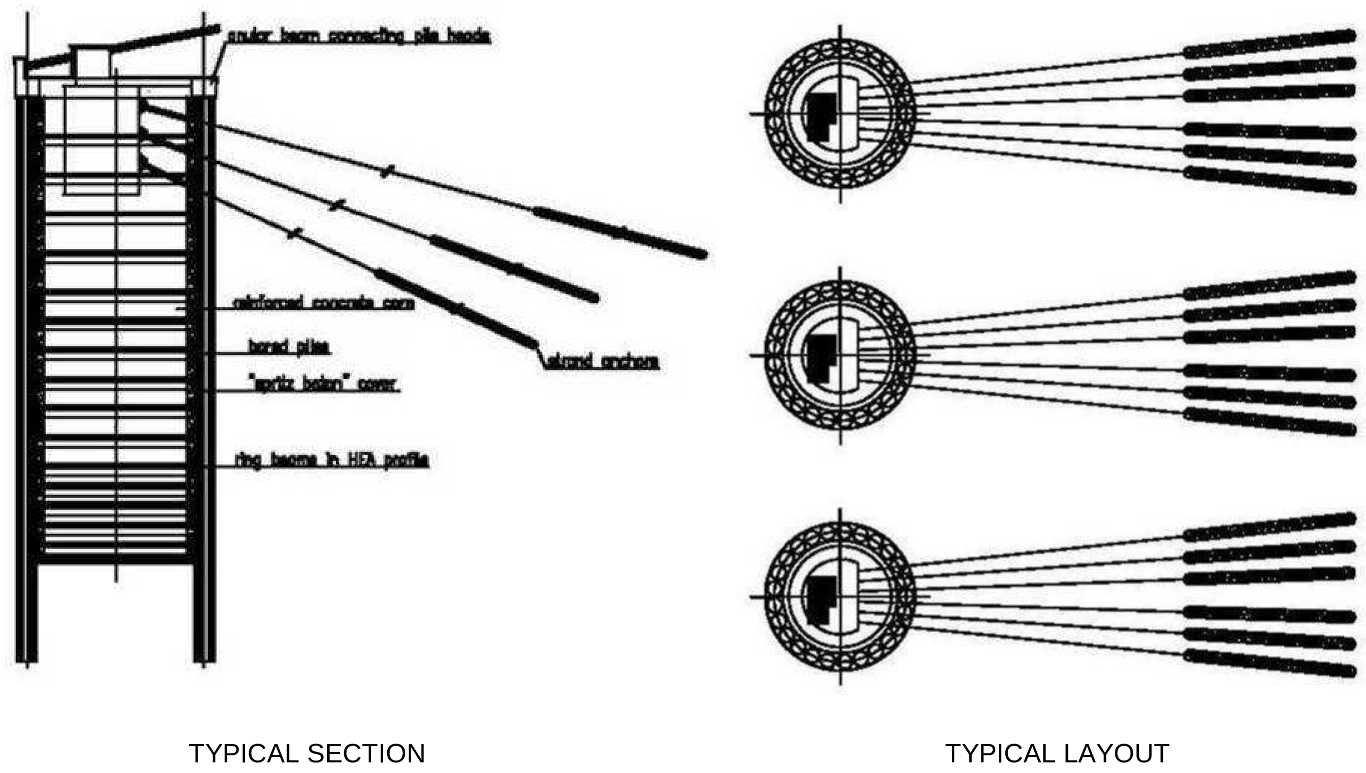
The calculation of the caisson capacity in relation to the soil/structure interaction may be carried out according to several approaches and simplified methods based on the simplified assumption that the caisson is infinitely rigid and is subject only to rotation (Pasqualini, 1975; Rocchi et al., 1992). A commonly used approach is that based on coupling the equation of global equilibrium with the deformations of the structure as determined using non linear spring; alternatively, soil- structure interaction analysis of horizontally loaded caisson may be carried out by 3D finite element analysis.

Finite element methods may be used instead to provide a simultaneous and consistent estimate of the soil-structure interaction both with the sliding mass and with the underlying stable soil. Finite element analyses in the time domain can also be used to refine the evaluation of the performance of the structure under seismic conditions.

The mechanical charateristics of the caissons must be adequate to sustain the actions and the effects of actions on them. The structural checks must satisfy all applicable codes and standards on the subject.

It is important that the designer considers the adequacy of the annular structureand of the stability of the temporary excavations, including consideration of base stability (reverse bearing capacity, piping, blow out). The methods of analysis must reflect the details of construction. It is prudent not to rely solely on the annular resistance of structures formed by adjacent vertical elements and the reduced annular stiffness of this type of construction compared to the axial stiffness of monolithic elements. Nonetheless, the structure needs to be designed to resist at-rest soil pressures.

Figure 2: Schematic plan and section (source: SGI-MI project files)



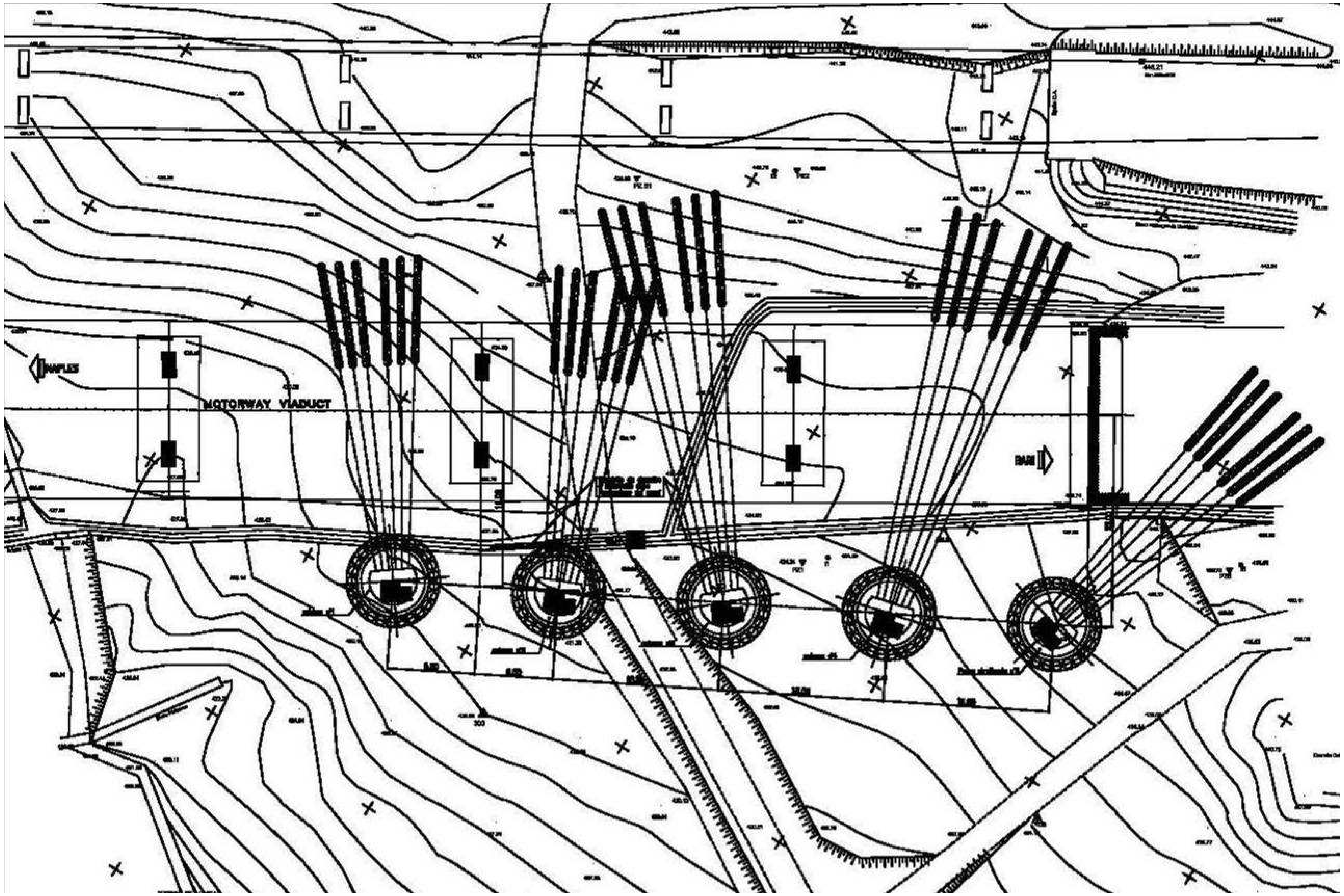
MITIGATION THROUGH REDUCTION OF HAZARD	
6 TRANSFER OF LOADS TO MORE COMPETENT STRATA	
6.4 CAISSONS – MECHANICAL EFFECTS	
Picture 1: Excavation with temporary retaining structure consisting of bored piles and concrete annular beams (source: SGI-MI project files) 	Picture 2: Caisson top - chamber for the ispection of strand anchor heads (source: SGI-MI project files) 
Picture 3: Construction during excavation by means of consecutive concrete rings (source: SGI-MI project files) 	Picture 4: Reinforcement rebars of the chamber walls sustaining the strand anchor heads (source: SGI-MI project files) 

MITIGATION THROUGH REDUCTION OF HAZARD

6 TRANSFER OF LOADS TO MORE COMPETENT STRATA

6.4 CAISSONS – MECHANICAL EFFECTS

Figure 2: Typical layout of structural caissons equipped with active strand anchors - see also schematic section in Figure 1 (source: SGI-MI project files)



MITIGATION THROUGH REDUCTION OF HAZARD

6 TRANSFER OF LOADS TO MORE COMPETENT STRATA

6.4 CAISSONS – MECHANICAL EFFECTS

APPLICABILITY

Class	Descriptor	Rating	Notes
Type of movement (Cruden & Varnes, 1996)	Falls	0	Best suited to slides and the slide-like portion of complex landslides. May be applicable in some cases to prevent the triggering of slides with the potential to turn to spreads or flows, but are substantially ineffective once fuidification has occurred.
	Topples	0	
	Slides	8	
	Spreads	4	
	Flows	4	
Material	Earth	8	Difficoult, very expensive and typically inappropriate in rock, but can be extended into rock if required. Method of construction to be selected taking into account ground and groundwater conditions.
	Debris	8	
	Rock	0	
Depth of movement	Superficial (< 0.5 m)	0	Typically: • best suited where the movement is deep (> 8 m, up to 20 – 25 m), • inappropriate in shallower movements because excessive.
	Shallow (0.5 to 3 m)	0	
	Medium (3 to 8 m)	4	
	Deep (8 to 15 m)	6	
	Very deep (> 15 m)	8	
Rate of movement (Varnes, 1978)	Moderately to fast	0	Workers' safety and end result require construction to take place when movement is extremely slow or very slow (maximum 1.5 m/year, corresponding to approximately 5 mm/day).
	Slow	2	
	Very slow	6	
	Extremely slow	8	
Groundwater	Artesian	2	High groundwater levels associated with coarse grained materials and/or artesian groundwater conditions require special dewatering during construction, possibly making this technique not feasible in extreme cases.
	High	6	
	Low	8	
	Absent	8	
Surface water	Rain	8	Water courses need to be temporarily diverted or reliably dry during construction. Potential pollution of watercourses by construction operations, especially for the first stage annular structure (for example by drilling fluid and/or by grout) may impose restriction on construction procedure. No problems once the works are completed, except possibly when caissons interfere with the banks of watercourses, modifying the erosion regime.
	Snowmelt	8	
	Localized	6	
	Stream	2	
	Torrent	0	
	River	0	
Maturity		8	Technique and design process are well established and widely used in suitable conditions.
Reliability		8	Reliable performance in well characterized landslides; in first time slides it depends on estimate of piezometric regime and apprporiate operational strength parameters of soil, which can be problematic.
Implementation		6	Requires specialist equipment and techniques; implementation may need temporary roads and working platform for safe operation.
Typical Cost		2	Very expensive.

Note
Ratings are given on a scale of 1 to 10; the higher the grade, the most suitable is the specific method under consideration to use in landslides of the given characteristics, evaluated individually. Overall suitability to specific case under consideration may be obtained by a weighted average of these ratings, with user defined weights. Zero rating means "not applicable"

MITIGATION THROUGH REDUCTION OF HAZARD	
6 TRANSFER OF LOADS TO MORE COMPETENT STRATA	
6.4 CAISSONS – MECHANICAL EFFECTS	
References: Brandl H. (1888). ”Stabilization of deep cuts in unstable slopes”. Proc. of 5th International Symposium on Landslides, Lausanne (Swiss), Balkema, 867-872. De Paoli B. (1989). ”La costruzione di fondazioni a pozzo alla luce dello sviluppo delle tecniche operative”. Proc. 17 Convegno Nazionale di Geotecnica, Taormina (Italia), AGI, 51-58, (in Italian). Leoni F., Manassero V. (2003). ”Consolidamento e rinforzo dei pendii in terra”. Proceedings of the 19th Conference of Geotechnics of Turin (CGT), Turin (Italy), (in Italian). Pasqualini E. (1975). ”Criteri di dimensionamento delle fondazioni a pozzo”. Proceedings of the 6th Conference of Geotechnics of Turin (CGT), Turin (Italy), (in Italian). Rocchi G., Collotta T., Masia A., Trentin M., Fittavolini C. (1992). ”Design aspects on piers foundations in landslip-prone seismic areas”. Proc. of 6th International Symposium on Landslides, Christchurc (New Zealand), Balkema, 1957-1969. Tambara F. (1999). ”Stabilizzazione dei pendii”. Hevelius editore, (in Italian).	

MITIGATION THROUGH REDUCTION OF HAZARD

6 TRANSFER OF LOADS TO MORE COMPETENT STRATA

6.5 SOIL NAILING

Description

Soil nailing is the insertion of solid or hollow steel or glass fibre bars into the face of an excavation or an existing slope to reinforce it, transferring part of the load from the potentially unstable mass to more competent strata, typically where the potentially unstable mass has a maximum thickness of 6 to 8 m. The face of the slope is protected by shotcrete and welded wire mesh, geogrid/geotextiles sheets and cast-in-place concrete or prefabricated panels.

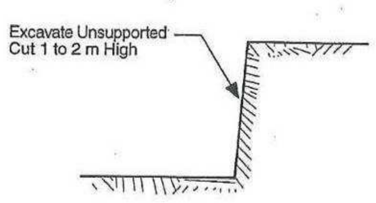
The technique has been developed in France, Germany and United States over the past 25 years or so (Guilloux and Schlosser, 1985; Nicholson, 1986; Bruce and Jewell 1986a; 1986b; Munfach et al.; 1987; Juran and Elias, 1987; Gnilsen, 1988, Recommendations Clouterre, 1991; Byrne et al., 1998; Mitchell and Jardine, 2002; Phear et al., 2005), as a development of the "root piles" technique originally developed in the 1950's described by Lizzi (1977); Bruce (1992a, b). Its application has extended to a wide variety of ground types, from soils to weathered and un-weathered rocks; while the term "ground nail" might be a more suitable generic term, "soil nail" has become established as the commonly accepted generic terminology and is used here for nails installed in all types of ground which can be conveniently described as continuoum. Case histories are listed for example in Bruce and Jewell (1987a; 1987b) and Bruce (1989).

A typical construction sequence for drilled and grouted nails is described below and shown in Figure 1; alternative methods of installations include percussive methods or vibro-drilling (Myles and Bridle, 1991), combinations of vibration driving with injections and driving nails by compressed air or pyrotechnic launchers A typical application is shown in Figure 2.

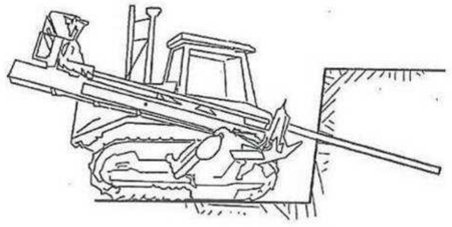
1. Installation of ditches to intercept and divert surface water; exacavation/trimming in stages of limited height (typically 1 or 2 m), minimizing ground disturbance and removing loosened areas, leaving a working bench of 5÷7 m width. For installation in existing slopes, special provision must be made for access (long reach booms, sledges or similar).
2. Dilling of nail holes at predetermined locations to a specified length and inclination using drilling methods appropriate for the ground, supporting the drillhole with casing, if required, although this will often have serious adverse impact on the cost effectiveness of soil nailing. Bentonite or other mud suspensions should not be used, as "smear" on the drillhole walls can significantly reduce the grout-to-ground bond. Typical drillhole size: 100 to 300 mm; spacing: 1 to 2 m, both vertically and horizontally; inclination: 15° below horizontal to facilitate grouting; length: 6 to 15 m (up to 28 m using large hydraulic-powered track-mounted rigs with continuous flight augers).
3. Installation and grouting of nails. Plastic or steel centralizers are commonly used to center the nail in the drillhole; stiffer grout mix may be alternatively used to maintain the position of the nail and prevent it from sinking to the bottom of the hole. The steel nails are commonly 25 to 50 mm in diameter; solid or hollow; the yield strength is 420 to 500 N/mm^2. Steel nail diameter smaller than 25 mm are not recommended due to difficulties associated with placement of such flexible tendons in drilled holes. Grouting takes place under gravity or low pressure from the bottom of the hole upwards. Grouted steel nails protected only by the grout annulus are not generally considered adequate for permanent application in some countries; in this cases, additional protection against corrosion may be given by sacrificial thickness, by heavy epoxy coating and by encapsulating it in a grout-filled corrugated plastic sheathing. "Self-drilling nails" (Figure 3) can be used where open hole drilling is not possible or practical. However, they require special corrosion considerations and testing procedures to be considered for permanent applications. In general the self-drilling nails should not be used in aggressive ground (as defined in Byrne et al., 1998) and coatings should not be considered acceptable corrosion protection, which can be assured only by providing sacrificial steel.
4. Placement of drainage system and installation of the construction facing and of the bearing plates. Prefabricated synthetic drainage mats are placed in vertical strips (about 400 mm wide) between the nail heads at horizontal spacing equal to that of the nails. The drainage strips are extended down to the base of the structure and connected either directly to a footing drain or to weep holes that penetrate the final wall facing. If water is encountered, short horizontal drains are generally required to intercept the water before it reaches the face. The construction facing typically consists of a mesh-reinforced shotcrete layer of the order of 100 mm thick. Following placement of the shotcrete a steel bearing plate (typically 200 mm x 250 mm square and 20 mm thick) and securing nut are placed at each nail head and the nut is hand wrench tightened sufficiently to embed the plate a small distance into the still plastic shotcrete.
5. Progressive construction to the final grade. In excavation or on large slopes, the process described at steps 1 to 4 is repeated in stages to the final grade. The maximum bench height and construction sequence must be verified carefully, to ensure stability at all stages of construction.
6. Final facing. For long term structural durability, a concrete facing or a second layer of shotcrete is finally applied on the exposed surface. Rip rap or biotechnological finishes also applied, especially in landslide stabilization works.

Figure 1: Schematic construction sequence (sketches after Byrne et al., 1998; photos by USA Corps. of Eng.s)

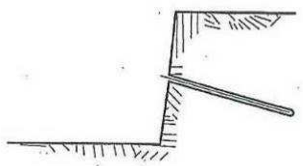
STEP 1
Install cut-off drainage and excavate unsupported cut, 1 to 2 m high



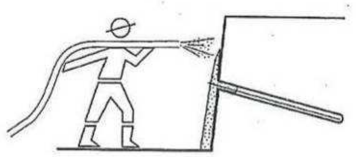
STEP 2
Drill hole for Nail



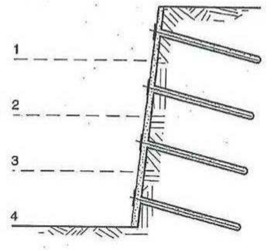

STEP 3
Install and grout Nail



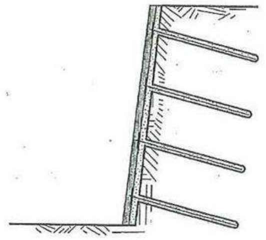

STEP 4
Place drainage strips, initial shotcrete layer and bearing plates and nuts




STEP 5
Repeat process to Final Grade



STEP 6
Place Final Facing
(on permanent walls)




MITIGATION THROUGH REDUCTION OF HAZARD

6 TRANSFER OF LOADS TO MORE COMPETENT STRATA

6.5 SOIL NAILING

In case of permanent reinforcement and use of drilling and grouting methods, the steel bar is encapsulated in a cement grouted body to provide corrosion protection and improved load-transfer to the soil; the steel bar is also typically protected with a heavy epoxy coating or by encapsulation in a grout-filled corrugated plastic sheathing. For other installation methods protection against corrosion can be provided by sacrificial thicknesses; BS 8006:1995 gives guidance on sacrificial thicknesses for galvanized and non-galvanised nails. When shotcrete facing is not adopted, corrosion protection at the nail head may be provided by precat or cast-in-place concrete head details.

Nails are characterized by "continuous" reinforcement with transfer of shear stress along the full length of the inclusion. The effect is to reduce nail forces at the face, allowing the use of only a thin cover, primarily to resist erosion or slump of the face. The nails are installed horizontally or suborizontally, approximately parallel to the direction of major tensile straining in the soil. The nails work predominantly in tension, but are considered to work also in bending/shear, especially where the orientation is perpendicular to the anticipated shear surface; in these cases nails may more properly be called dowels.

The nails contribute to the support of the soil partially by directly resisting the destabilizing forces and partially by increasing the normal loads (and hence the shear strength) on potential sliding surfaces (see Figure 3 of fact sheet 6.0 on the general aspects of hazard mitigation by transfer of load to more competent strata). The reinforcements are passive and develop their action through nail-soil interaction as the soil deforms; the face protection need to be installed in order to keep the soil from caving in between the bars.

The reinforced soil body (nails plus face protection) becomes the primary structural element; in fact, the reinforced zone performes as a homogeneous resistant unit to support the unreinforced soil behind it in a manner similar to a gravity wall. (Stocker et al., 1979).

The technique offers several advantages:

- Construction flexibility in heterogeneous soils with cobbles, boulder and other hard inclusions, as the obstructions offer no problems for the relatively small diameter nail drillholes.
- Well suited to sites with difficult or remote access because of the relatively small size and mobility of the equipments.
- High system redundancy as the soil nails are installed at high density and the consequence of a unit failure are therefore correspondingly less severe.
- The system is relatively robust and flexible and can accommodate significant total and differential displacements.
- Soil nailing has been documented to perform well under seismic loading conditions (See for example Felio et al., 1990).
- Additional nails can easily be installed during construction, if slope movements occur or is greater than expected.
- The method is well suited for rehabilitation of distressed retaining syructures.

The disadvantages of the technique are mainly linked to its constructability, in relation to nature of ground to be reinforced and/or presence of groaundwater percolating through the face; in general, the economical use of soil nailing requires that the ground be able to stand during construction. In addition, when the drill and grout methods is adopted, it is highly desirable that the open drillhole can maintain its stability for at least several hours. Therefore difficulties can be experienced in:

- Loose clean sands and gravels or coarse grained soils of uniform size unless in a very dense condition; these soils will not generally exhibit adequate stand-up time and are also sensitive to vibration induced by construction equipments.
- Soils with excessive water content or below the groundwater; significant groundwater seepage at the exposed face can cause serious problems (e.g. local slump; drillhole instability, impossibility to obtain a satisfactory ground-grout bond).
- Organic soils or clayey soils with Liquidity Index greater than 0.2 and undrained shear strength less than 50 kPa; remoulding caused by nail installation in may reduce skin friction to unacceptable values.
- Higly fractured rocks with open joints or voids and open graded coarse materials (e.g. cobbles), geotextile nail socks or low slump grout may be necessary in such materials to mitigate the difficulty of satisfactorily grouting the nails.
- Rock or decomposed rock with weak structural discontinuities inclined steeply toward and daylighting into the cut face.
- Expansive (e.g. swelling) soils; these soils may result in significant increases in the nail loading near the face. Water must be prevented from reaching expansive soils that are soil nailed.

It should also be noted that the long-term performance of shotcrete facings has not been fully demonstrated, particularly in areas subjected to freeze-thaw cycles. In these circumstances it is recommended that the design prevents frost from penetrating the soil by provision of an appropriate protective structure (e.g. granular or synthetic insulating layer).

Special attention must be paid in both the design and the construction stage to the issue of corrosion and durability of the structural elements. For further guidance on this issue, reference may be made to Recommendations Clouterre (1991), Phear et al. (2002) and Byrne et al. (1998), who also provides detailed recommendations on drainage and frost protection.

Figure 2: Typical section of soil nailing for slope remediation (source: SGI project files)

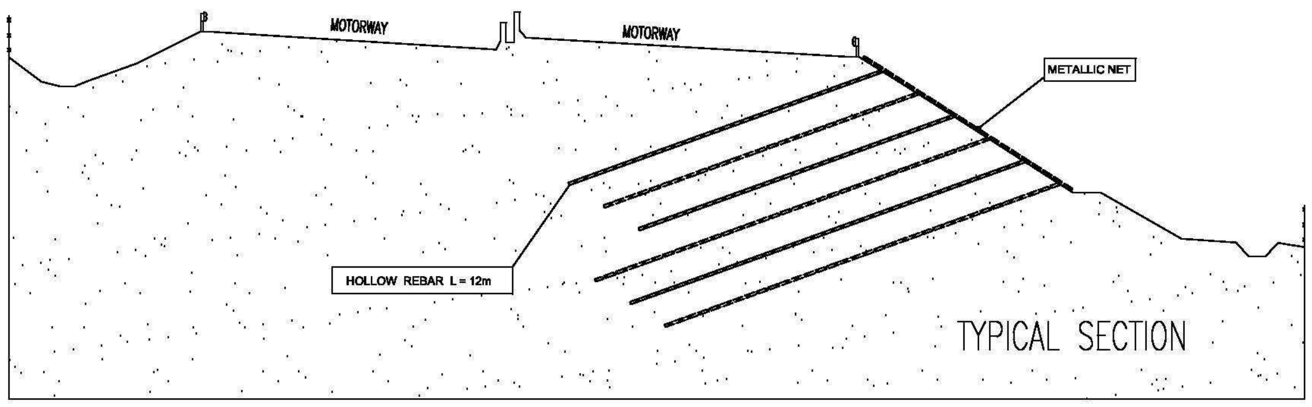
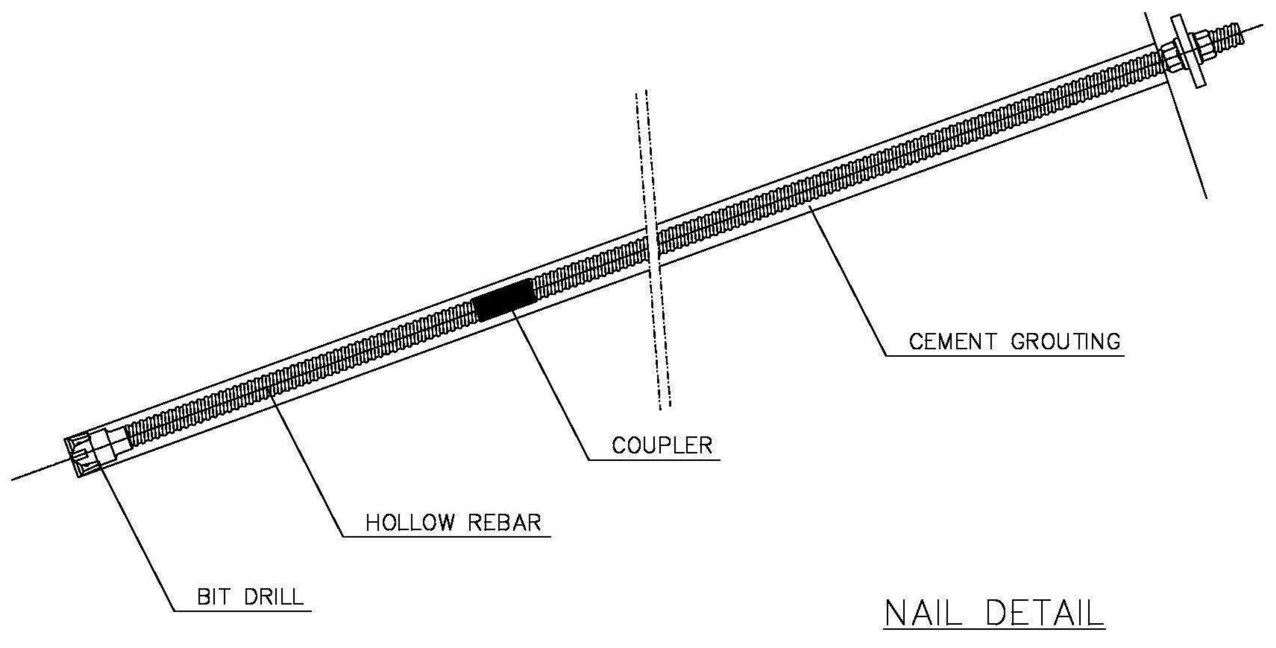


Figure 3: schematic detail of self-drilling hollow soil nail (source: SGI project files)



MITIGATION THROUGH REDUCTION OF HAZARD

6 TRANSFER OF LOADS TO MORE COMPETENT STRATA

6.5 SOIL NAILING

Design

As highlighted by Mitchell and Jardine (2002), there is still much discussion about the assessment of the behavior and stability of nailed structures. A discussion on the differences between the design approaches widely used in Europe and the United States, as reported in Schlosser (1983) and Juran and Beech (1984), can be found in Juran and Elias (1987), Gnilsen (1988), Jewell (1990), Jewell and Pedley (1990a; 1990b; 1990c; 1991), Bridle and Barr (1990) and Schlosser (1991).

Solutions to the problem require:

- Carrying out appropriate soil structure interaction analyses to investigate the internal stability of the composite system made up by nails, facing and soil, both in the "active zone" close to the facing, where the shear stresses exterted by the soil on the reinforcement are directed outward and tend to pull the reinforcement out of the ground, and in the "resistant zone", where the shear stresses are directed inward and tend to restrain the reinforcement from pulling out.
- Evaluation of the overall stability of the nailed structure, considered as a massive retaining structure (external stability).

For internal stability to be achieved, the nail tensile strength must be adequate to provide the support force to stabilize the active block. The nails must also be embedded a sufficient length into the resistant zone to prevent a pullout failure.

In addition, the combined effect of the nail head strength (as determined by the strength of the facing or connection system) and the pullout resistance of the length of the nail between the face and the slip surface must be adequate to provide the required nail tension at the slip surface (interface between active and resistant zones).

All potential failure modes, which involve: a) face failure (active zone slides off the front of nails); b) pullout of nails from the resistant zone; c) structural failure of nails (in tension, bending or shear), must be analysed separately (simplified procedures) or simultaneously (advanced approaches).

Major difficulties in finding rigorous and reliable solutions for the internal stability of the nailed structure derive from the fact that both the forces acting in the nails and the forces acting on the facing are governed by the deformation behavior of the entire system, which, in turn, depends on the geometric and mechanical characteristics of the various elements (including the soil), together with the sequence, rate and method of construction. For example, the latter may influence the load transfer characteristics between soil and nail. The building of soil nailed structure involve a critical phase with respect to internal or external stability, which can be lower during the building phase than when the reinforcement is finally built. Therefore, internal and external stability of the nailed structure shall be checked for all the construction phases (Figure 4).

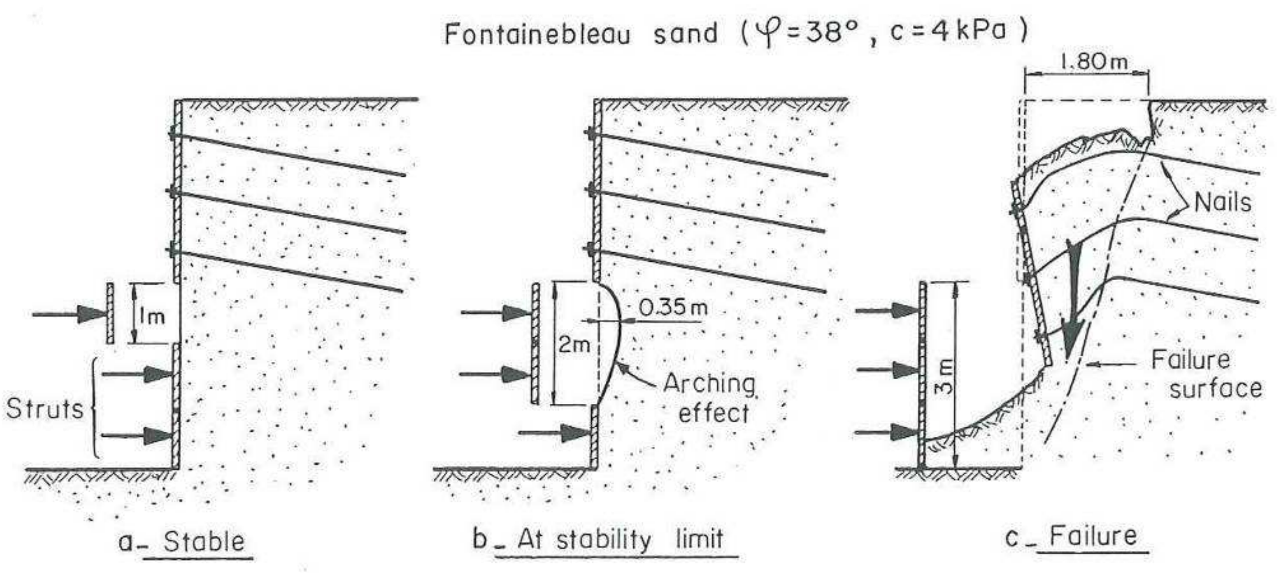
The simplest and most widely adopted method to investigate both internal and external stability of soil nailed structures is based on the slip surface limit equilibrium method by incorporating the reinforcing effect of the nails, including consideration of the strength of the nail head connection to the facing, the strength of the nail tendon itself and the pullout resistance of the nail-ground interface. Typically, the analyses are carried out with reference to Ultimate Limit States, with the magnitude of deformations (Servicibility Limit States) controlled indirectly by application of appropriate values of partial factors in ULS calculations. Where deformations are critical, it becomes necessary to resort to numerical analyses.

The contribution of any nail to the stability of a particular sliding surface will be the least of a) the tensile strength (shear/bending contributions neglected) or the "ideal" strength (shear/bending contributions considered) of the nail; b) the pullout resistance of the length of nail beyond the slip surface; c) the nail head strength plus the pullout resistance of the length of nail between the slip surface and the face of the exposed surface. All potential surfaces must be examined to ensure that the design is complete.

The potential contribution of shear and/or bending of the nails to the overall resistance of the system is typically negligible and in any case difficult to evaluate, with different procedures being proposed in the literature (Schlosser, 1982; Schlosser, 1983; Blondeau et al., 1984; Jewell and Pedley, 1990a, b; Juran et al. 1990; Schlosser, 1991). Experimental studies (for example Jewell and Pedley, 1990a, b) have shown that this contribution is less than 10% of that provided by tensile forces and is only achieved after large displacements, as also stated by Gässler (1990).

In case of drill and grout method of nail installation, the pullout resistance of the nail will be the least of ground-grout bond and grout-tendon bond. Ground-grout bond is strongly dependent on the method of construction; for this reason both pullout tests and short-term creep tests are a standard part of nail preliminary testing for check and calibration of the design before starting with the construction activities; in short-term creep tests, the rate of creep of the nail will increase as the applied load increases; a creep rate exceeding 6 mm/60 minutes is generally considered unacceptable (see for example Byrne et al., 1998).

Figure 4: Stability of excavation phase in CEBTP No.2 experimental wall (after Recommendation Clouterre, 1991)



Picture 1: Nailing on embankment slope using self-drilling hollow bars (source: SGI project files)

MITIGATION THROUGH REDUCTION OF HAZARD

6 TRANSFER OF LOADS TO MORE COMPETENT STRATA

6.5 SOIL NAILING

For preliminary design evaluation of ground-grout bond, reference can be made for example to Bustamente and Doix (1985). More generally, reference may be made to the charts proposed by Recommendation Clouterre (1991) which relate pullout resistance to the type of soil and the method of installation, subject always to verification by pullout tests in the field.
Pullout tests can also be carried out in the laboratory, but the boundary conditions of the apparatus and the idealization of the field conditions mean that the results from such tests are not always realistic.
In case of continuous threadbars, grout-tendon bond is typically an order of magnitude or more higher than the ground-grout bond and is therefore not critical for soil nailing applications when proper grout mix and installation techniques are used.
The strength of the nail head may be controlled by the flexural and punchning shear strength of the facing; these strengths are usually determined by specific structural analyses, taking into account the grid layout of the nails; some examples are given by Byrne et al. (1998) and by Phear et al. (2005). Other potential failure mechanisms do exist for the nail head; however,
these modes will not usually control the design or limit the nail head strength for the types of systems commonly employed in soil nail structure construction. For discontinuous facing elements, the face plate should be checked against bearing failure (see DoT Advice Note HA 68/94, 1994 for guidance). External stability refers to the potential deformation modes typically associated with gravity or cantilever retaining structures and involves considerations of:

- Horizontal sliding and/or overturning under the lateral earth pressure of the ground retained behind the reinforced mass.
- Bearing capacity failure under the combined effect of self weight and lateral earth pressure loading.
- Overal slope stability of the ground on which the soil nailed structure is located.

In the simplified procedure, both internal and external stability analyses are usually carried out in 2D (plane strain) conditions.
In order to check both stability and deformation behaviour of the soil nailed structure the analyses carried out with the simplified procedure can be supplemented by true soil-nail-facing interation analyses with the use of finite element (FE) methods; the best approach is to use 3D models, where the nail is modelled explicitly as is; often the 3D geometry is such that it can be simplified considering symmetry in the model.
In static conditions, the reliability of the design method depends on the correct selection of the operational strength parameters of the soil and on the correct modelling of the ground-grout load transfer curves; uncertainties can be minimized by preliminary pull-out tests.
The internal and external stability under seismic conditions can be investigated by means of pseudo-static methods and/or finite element methods; the external stability can be also investigated by means of Newmark type of analysis. The reliability of pseudo-static analyses depends on the same factors affecting static analyses, with the addition of uncertainties on the appropriate values of pseudo-static seismic coefficient kh to be used; Newmark type analyses must be carried out for a large number of strong motion records and the results must be treated by statistical techniques to minimize error.
FEM analyses retain all the limitations of the simpler methods, except that they can incorporate a more detailed constitutive modelling of soil behaviour, overcoming the need to preselect operational values of strength, as well as geometric simplifications.
Systematic monitoring and reporting of performance is necessary, both to verify that the structure performs as anticipated and to enhance confidence and expertise in the use of this technique in the future, especially in light of continuing debate on the best methods of design. In particular, monitoring of any lateral outward movement of the face is highly desirable. Designers should detail monitoring requirements (type, location, frequency and data treatment) as an integral part of the design.
Performance monitoring instrumentation should include slope inclinometers, survey points and nail loads at the head and along the nail length to measure movements and stresses during and after construction.
Sufficient environmental monitoring should also be carried out to provide the necessary framework for interpretation of performance monitoring. Environmental monitoring should include, as a minimum, temperatuire variations and groundwater levels.
Monitoring should continue for a period of at least 2 years after construction, in order to gather information as a function of time and environmental changes such as freeze-thaw cycles and/or variations in groundwater levels.
For further details on the design of soil nailing stuctures, reference may made to the guidelines published in France (Recommendation Clouterre, 1991); the United Kingdom (DoT, 1994; BSI, 1995; Phear et al., 2005) and the United States (Byrne et al., 1998; Lazarte et al., 2003).

Picture 2: nailing on debris slope using nails grouted in predrilled holes; note associated wire mesh and biotechnical facing (source: SGI project files)

Picture 3: nailing on debris slope using nails grouted in predrilled holes; note associated wire mesh and biotechnical facing (source: SGI project files)

MITIGATION THROUGH REDUCTION OF HAZARD			
6 TRANSFER OF LOADS TO MORE COMPETENT STRATA			
6.5 SOIL NAILING			
APPLICABILITY			
Class	**Descriptor**	**Rating**	**Notes**
Type of movement (Cruden & Varnes, 1996)	Falls	6	Applicable to slides and in special circumstances to falls and topples in cemented or stiff/hard cohesive soils.
	Topples	6	
	Slides	8	
	Spreads	0	
	Flows	0	
Material	Earth	8	Applicable to earth and debris. In very coarse debris drilling can be problematical and launching is precluded.
	Debris	6	
	Rock	0	
Depth of movement	Superficial (< 0.5 m)	8	Practical soil nail lengths and the need to achieve sufficient anchorage in the underlying stable soil limit the application of this technique to situations where the residual thickness of the actual or potential landslide to be stabilized is significant.
	Shallow (0.5 to 3 m)	8	
	Medium (3 to 8 m)	6	
	Deep (8 to 15 m)	0	
	Very deep (> 15 m)	0	
Rate of movement (Varnes, 1978)	Moderately to fast	0	Workers' safety and end result require construction to take place when movement is extremely slow or very slow (maximum 1.5 m/year, corresponding to approximately 5 mm/day). Under special conditions and taking due precautions it may be carried out when movement is "slow" (up to 1.5 m/month, corresponding to 5 cm/day) .
	Slow	2	
	Very slow	8	
	Extremely slow	10	
Groundwater	Artesian	0	Drillhole stability where groundwater may be encountered should be reviewed carefully, since the use of temporary casing, if required, would normally make this technique excessively expensive. Groundwater seepage at the surface must be avoided, incorporating suitable drainage works, with the risk of local slumping before the draingae works are effective.
	High	2	
	Low	4	
	Absent	10	
Surface water	Rain	8	Where sliding is due to channelized water, construction difficulties may be expected and there may be special requirements for the facing.
	Snowmelt	8	
	Localized	4	
	Stream	2	
	Torrent	0	
	River	0	
Maturity		6	There is over 25 years experience with the technique, but it is still susceptible to technological and design improvements.
Reliability		6	Successful application depends on correct schematization and characterization of the landslide, design and construction detail, correct application.
Implementation		6	Requires specialist equipment; special arrangements may be required for access on existing slopes; simplified by launching but durability is questionable.
Typical Cost		6	Moderate. Can become quite high if drillholes require temporary casing and/or special access arrangements.

Note
Ratings are given on a scale of 1 to 10; the higher the grade, the most suitable is the specific method under consideration to use in landslides of the given characteristics, evaluated individually. Overall suitability to specific case under consideration may be obtained by a weighted average of these ratings, with user defined weights. Zero rating means "not applicable"

MITIGATION THROUGH REDUCTION OF HAZARD

6 TRANSFER OF LOADS TO MORE COMPETENT STRATA

6.5 SOIL NAILING

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MITIGATION THROUGH REDUCTION OF HAZARD

6 TRANSFER OF LOADS TO MORE COMPETENT STRATA

6.6 DOWELS AND HARNESSING

Description

Dowels are short untensioned steel bars inserted and grouted into holes drilled across the potentially unstable block or slab down to the underlaying stable rock; they are usually about 25 mm in diameter, embedded 0.5 to 1.0 m into the sound rock below and spaced about 0.5 to 0.8 m apart. A typical example is shown in Picture 1.

Dowels are generally adopted in situations characterized by:

- presence of isolated potentially unstable blocks or slabs of rock located on an otherwise stable slope of parent rock, with clearly identifiable discontinuities separating the potentially unstable mass from the underlying stable slope;
- situations where the removal of the potentially unstable mass (scaling and trimming) is impractical, for example because it would interfere unacceptably with existing structures or infrastructure.
- situations where geomorphological conditions and/or the presence of structures or infrastructure at the toe of the slope do not allow the installation of passive barriers.

Dowels are installed approximately perpendicular to the sliding surface, to provide additional shear resistance across the potential failure surface. They are used to support blocks or slabs of rock with thicknesses up to 1 to 2 m. They are most effective when there has been no prior movement of the rock so that there is interlock on the potential sliding surface. For blocks or slabs thicker than 1 to 2 m or where there has been previous movement, the required support may be provided more reliably by rock bolting (un-tensioned or tensioned)

When istalled at the toe of the block or slab, dowels are provided by a cap of reinforced concrete which encases the exposed steel; in these cases the concrete shall be in contact with the rock face so that movement and loss of interlock on the potential rupture surface are minimized.

Where the mass to be supported is fractured into blocks which are too small to be dowelled individually and/or rests on material which is not sufficiently competent to provide adequate anchorage to the dowels, the potentially unstable mass may be harnessed by structural netting (or, more rarely, ropes) of adequate stiffness and resistance, anchored by dowels along the edges of the potentially unstable mass. A typical example is shown in Picture 2.

Design

Dowels operate on the following basic principles:

- The dowels restrict movement along the potential failure plane so as to preclude possible reductions in available resistance that could arise from loss of interlocking;
- Sufficient additional shear strength is provided by the dowels at small deformation such that, together with the resistance already available (and preserved) along the discontinuity, sufficient overall resistance is provided to guarantee the stability of the potentially unstable mass with an adequate factor of safety.

The objective of the design is to define the number and charateristics of dowels necessary to ensure that the principles decribed above are satisfied.

The additional shear resistance to be provided by the dowels in static and seismic conditions may be evaluated on the basis of planar and/or wedge limit equilibrium analyses of the type amply discussed in fact-sheet 2.0 on “General aspects of mitigation by changes to slope geometry and/or mass distribution”.

The additional shear resistance provided by the dowels with respect to a specific discontinuity and dowel configuration may be evaluated for example as proposed by Panet (1987), taking into account both the shear resistance of the dowel and the additional resistance associated with the tension which is induced in the dowel by dilatancy on the discontinuity and/or by geometrical effects. For simplicity, these additional contributions may be ignored, considering the shear resistance of the dowel alone, especially where the dowels are close to perpendicular to the potential failure surface.

Where dilatancy along the discontinuity and/or geometrical effects are taken into account, the length of embedment in the potentially unstable mass and especially in the underlying stable material must be sufficient to provide adequate longitudinal anchorage to the dowel.

The results of the analyses depend critically on the precise modelling of the geometry of the discontinuities which represent the potential failure surfaces, as well as the shear resistance and dilatancy along the potential failure surface.

In order for the dowels to operate as anticipated, both the potentially unstable mass and the underlying stable material must provide sufficient lateral resistance to the dowel, which is normally the case in competent rock but may need to be verified in highly weathered rock, weak rocks and/or rocks susceptible to weathering.

The design should include careful consideration of access and operating conditions and the associated safety precautions.

Picture 1: Dowels to stabilize specific blocks above an existing railway (source: SGI-MI project files)

Picture 2: Harnessing to stabilize a group of smaller blocks above an existing railway (source: SGI-MI project files)

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6.6 DOWELS AND HARNESSING			
APPLICABILITY			
Class	**Descriptor**	**Rating**	**Notes**
Type of movement (Cruden & Varnes, 1996)	Falls	8	Typically most suitable to prevent sliding of individual blocks; in special circumstances may be used also to prevent rotation/toppling of individual blocks.
	Topples	2	
	Slides	0	
	Spreads	0	
	Flows	0	
Material	Earth	0	Requires both potentially unstable mass and underlying stable material to be competent rock; harnessing when potentially unstable mass is highly fractured.
	Debris	0	
	Rock	8	
Depth of movement	Superficial (< 0.5 m)	8	Typically suitable for blocks/slabs up to 1 to 2 m depth only.
	Shallow (0.5 to 3 m)	2	
	Medium (3 to 8 m)	0	
	Deep (8 to 15 m)	0	
	Very deep (> 15 m)	0	
Rate of movement (Varnes, 1978)	Moderately to fast	0	Bloks must be stable at time of construction.
	Slow	0	
	Very slow	0	
	Extremely slow	8	
Groundwater	Artesian	0	Suitable for all groundwater conditions; ”artesian” not applicable to the type of situation treated by dowels and harnessing.
	High	6	
	Low	6	
	Absent	6	
Surface water	Rain	8	Not practical within or close to water courses.
	Snowmelt	8	
	Localized	6	
	Stream	0	
	Torrent	0	
	River	0	
Maturity		8	Well established technique, widely used where applicable. Often insufficent attention paid to durability.
Reliability		8	Simple schematization and analysis. Possible pitfalls in the systematic identification of blocks or slabs to be treated.
Implementation		6	Requires access on steep slopes, implying specialist equipment. Works must be planned carefully to avoid exposing workers to rockfall from above.
Typical Cost		6	Typically moderate; access conditions may have a strong impact on cost.

Note
Ratings are given on a scale of 1 to 10; the higher the grade, the most suitable is the specific method under consideration to use in landslides of the given characteristics, evaluated individually. Overall suitability to specific case under consideration may be obtained by a weighted average of these ratings, with user defined weights. Zero rating means ”not applicable”

MITIGATION THROUGH REDUCTION OF HAZARD	
6 TRANSFER OF LOADS TO MORE COMPETENT STRATA	
6.6 DOWELS AND HARNESSING	
References: Panet M. (1987). ”Reinforcement of rock foundations and slopes by active and passive anchors”. Proc. 6th Int. Conf. on Rock Mechanics, Montreal, Balkema, 1411-1420.	

MITIGATION THROUGH REDUCTION OF HAZARD

6 TRANSFER OF LOADS TO MORE COMPETENT STRATA

6.7 ROCK BOLTING

Description

Rock bolting is the systematic reinforcment and/or anchorage of rock slopes by the insertion and grouting of steel bars into holes predrilled into the more or less fractured rock mass, improving its stability. The deformed steel bars are typically 25 to 50 mm in diameter and up to 12 to 15 m in length. Long bolts are typically formed by joining shorter threaded bars using special couplers, to facilitate handling. For convenience of installation, strand anchors (see fact-sheet 6.8) are normally used where longer bolts are required. Bolts are installed across the discontinuities or the potential failure surfaces at a dip angle flatter than the normal and typically work mainly in tension and only subordinately in shear and bending.

Typically, drillholes in rock are self supporting. However, critical drilling conditions with potential loss of borehole stability may be encountered when drilling through higly fractured or milonitic zones, especially if water is also encountered in the drillhole. In this case, it may be simpler to grout and redrill the hole, rather than using a casing.

In relation to the degree of relaxation or loosening of the fractured rock to be reinforced and/or to be tied to the more competent rock below the bolts can be un-tensioned or tensioned. Relaxation and or loosening of the rock mass is a process that takes place as a results of unloading and weathering; once relaxation or loosening has been allowed to take place there is a loss of interlock between the blocks of rock and a significant decrease in the shear strength along the discontinuities and in the rock mass as a whole. Once relaxation or loosening has taken place, it is not possible to reverse the process. For this reason:

- where the degree of relaxation or loosening is relatively modest, it is possible to use passive (untensioned) rock bolting acting as pre-reinforcement (Moore and Imrie, 1982; Spang and Egger, 1990); the deformations necessary to activate the bolts are sufficiently small not to result in a significant reduction of the shear strength characteristics of the discontinuities and of the rock mass as a whole;
- where significant relaxation and loosening have already taken place, it may be necessary to install tensioned bolts in order to prevent further displacements and loss of interlock.

The advantages of using un-tensioned bolts are the lower costs and quicker installation compared with tensioned bolts.

From a conceptual point of view, un-tensioned (passive) rock bolts work in the same way as nails of soil nailing structures (see Figure 3 of fact sheet 6.0 on the “General aspects of hazard reduction by transfer of loads to more competent strata”). They are grouted for their full length in a single operation both below and above the potential failure surface. In slope applications, where the drillhole dips into the ground, there is no need for anchoring the distal end of the bolt. Even though in many situation a head plate is not strictly required, a end plate is normally fitted to the bolt at the surface and this may be usefull to anchor netting and or other facings that may be required.

From a conceptual point of view, tensioned (active) rock bolts work like anchors in tieback retaining structures (see Figure 2 of fact sheet 6.0 on the “General aspects - transfer of loads to more competent strata”). They are characterized by a anchor head, a free-stressing length and a bond length, located beneath the discontinuity or the potential failure surface.

Tensioned (active) bolts must satisfy three basic requirements:

1. There must be a suitable method of anchoring the distal end of the bolt in the drill hole;
2. A known tension must be applied to the bolt without creep and loss of load over time;
3. The complete bolt assembly must be protected from corrosion for the design life of the project.

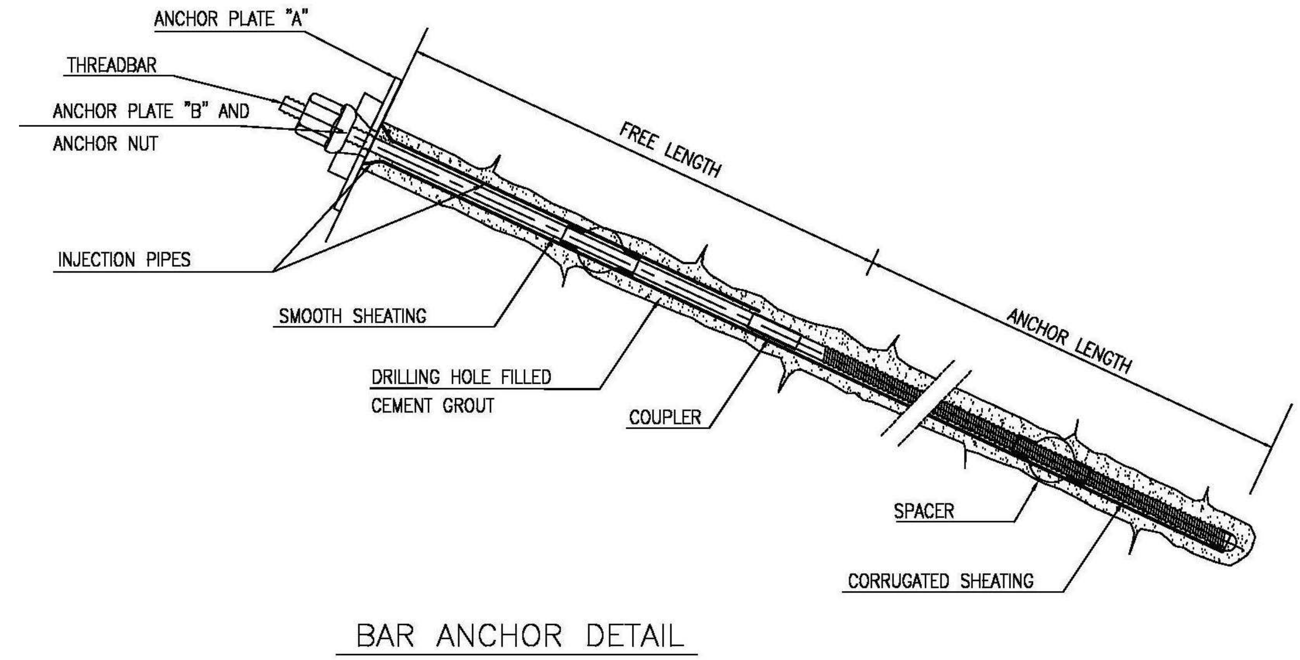
Methods of securing the distal end of a bolt in the drill hole include mechanical devices, resin and cement grout. The selection of the appropriate method depends on several factors such as the required capacity of the bolt, speed of installation, strength of the rock in the bond zone, access to the site for drilling and tensioning equipment and the level of corrosion protection required (Wyllie and Norrish, 1996).

The most appropriate method to ensure that bolts are not susceptible to creep and loss of load over time is to set operating loads significantly lower than the pullout resistance and below the level at which significant creep or fluage is observed in load tests. Specific test procedures have been developed for example by the Post Tensioning Institute (1985) and by AICAP (1993), which can detect the essential aspects of the behaviour of the anchor and the surrounding ground, to determine also the long term pullout resistancet rather than the short term resistance only.

Methods of protecting steel against corrosion include galvanizing, applying an epoxy coating and encapsulating the steel in cement grout. Because of the brittle nature of the grout and its tendency to crack, particularly when loaded in tension and in bending, the protection system is usually composed of a combination of grout and a plastic sleeve.

Alternatively, fibre reinforced polymer (FRP) bolts may be used to overcome problems of durability. Zhang et al (2001) provide useful guidance on these products. Up to date details may be obtained from manufacturers.

Figure 1: Schematic detail (source: SGI-MI project files)



Picture 1: Examples of bolts fitted with mechanical devices for securing the distal end (source: http://antech-tfa.com/en/3.2_stress.html)

MITIGATION THROUGH REDUCTION OF HAZARD

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6.7 ROCK BOLTING

Figure 1 shows a typical example of a three-layer corrosion protection system, where the bolt is encapsulated in a grout-filled HDPE sheat, and the outer annular space between the sheat and the rock is filled with a second grout layer, with centering sleeves to ensure complete encapsulation of the steel.
Grout mix can be readily pumped down a small-diameter grout tube, so that grouting proceeds from the distal end of the drill hole towards the surface, displacing any water or debris and producing a continuous grout column. Grouting is continued until clean grout flows out of the hole at the surface. Hollow bars can be used in lieu of soild bars, in which case the grout is injected through the bar itself, avoiding the need for the grout tube.
When bolting is carried out in a unweathered rock mass with relatively widely spaced discontinuities, the spacing between bolts may be commensurably wide and there is no need for any facing. In this case the end of the bolt is fitted with a small steel plate, typically embedded in a small concrete slab for corrosion protection (See Pictures 2 and 3 for an example).
Where the rock mass is highly fractured and/or the fractured rock may degrade and ravel from under and in between the reaction plates of the bolts, a structural facing must form an integral part of the rock bolting scheme. Different solutions may be foreseen for the structural facing, including for example:

- Reinforced concrete walls: the wall acts both as a protection against raveling of the rock and as a large reaction plate for the rock bolts; the rock bolt will be drilled through sleeves in the concrete; it is also important that there be drain holes through the concrete to prevent buildup of water behind the wall.
- Shotcrete, reinforced with reinforcing mesh (typically steel, but other materials may be equally suitable).
- Reinforced wire mesh, with a network of steel cables.
- Reinforced wire mesh associated with reinstatement of vegetation.

Design
Except for the specific differences that derive from the different nature of the material, the design of un-tensioned (passive) rock bolts is governed by much the same principles and rules as described in fact-sheet 6.5 on “Soil Nailing”, while the design of tensioned (active) rock bolts is governed by much the same principles and rules as described in fact-sheet 6.8 on “Strand anchors”. In particular, the following differences are noteworthy:

- from the point of view of the stability analyses used to determine the design load capacity and length of the inclusions, rock bolting deals with a discontinuous rock mass whose stability is typically governed by the discontinuities, as opposed to the pseudo-continuous nature of the ground involved in soil nailing schemes; appropriate methods of analyses, such as deterministic or probabilistic wedge analysis need to be applied;
- the grout-ground bond that can be developed in rock bolting (other than in argillaceous rocks) is typically much higher than is available in soil nailing, with an impact both on the minimum length of embedment beyond potential failure surfaces and on the lower demand on the facing; in limit cases, no facing at all will be required;
- the much greater stiffness of rock compared to soil allows the component of resistance associated with bending and shear to develop at much smaller displacements.

For tensioned cement-grout bolts the stress distribution along the bond length is higly non-uniform; the highest stresses are concentrated in the proximal end of the bolt immediately below the discontinuity or the failure surface, while ideally the distal end is unstressed (Farmer, 1975; Aydan, 1989). In practice the required length of the bond zone can be calculated with the simplifying assumption that the shear stresses at the rock-grout interface is uniformly distributed along the bond length. Limit values of the shear stresses can be estimated as a fraction of the uniaxial compressive strength of the rock in the bonded zone (Littlejhon and Bruce, 1975); allowable bond stresses related to rock strength and rock type are found in Wyllie (1991).
The diameter of the drillhole is determined by the available drilling equipment but must also meet certains design requirements. The hole diameter should be large enough to allow the bolt to be inserted in the hole without driving or hammering and be fully embedded in a continuous column of grout; a hole diameter significantly larger than the bolt will not improve the design and will result in unnecessary drilling costs and excessive grout shrinkage. A suitable ratio between the diameter of the bolt and the diameter of the hole is in the range of 0.4 to 0.6.
The working shear strength of the steel-grout interface of a deformed bar is usually greater than the working strength of the rock-grout interface; hence the length of the bond zone is typically determined from the stress level of the rock-grout interface.
Littlejohn and Mothersille (2008a; 2008b) provide guidance on issues related to maintenance and monitoring.

Picture 2: Drilling for installation of rockbolts on Polk County US-64 Rockslide
(source: http://news.tennesseeanytime.org/taxonomy/term/39)

Picture 3: Rockbolts on Polk County US-64 Rockslide after completion
(source: http://news.tennesseeanytime.org/taxonomy/term/39)

MITIGATION THROUGH REDUCTION OF HAZARD

6 TRANSFER OF LOADS TO MORE COMPETENT STRATA

6.7 ROCK BOLTING

APPLICABILITY

Class	Descriptor	Rating	Notes
Type of movement (Cruden & Varnes, 1996)	Falls	8	Typically most suitable to prevent widespread sliding and toppling on competent rock masses whose behaviour is governed by discontinuities.
	Topples	8	
	Slides	0	
	Spreads	0	
	Flows	0	
Material	Earth	0	Requires both potentially unstable mass and underlying stable material to be competent rock.
	Debris	0	
	Rock	8	
Depth of movement	Superficial (< 0.5 m)	6	Typically suitable to stabilize multiple slabs or wedges up to 8 m depth; requires additional facing where superficial instability occurs.
	Shallow (0.5 to 3 m)	8	
	Medium (3 to 8 m)	6	
	Deep (8 to 15 m)	0	
	Very deep (> 15 m)	0	
Rate of movement (Varnes, 1978)	Moderately to fast	0	Rock face must be stable at time of bolting.
	Slow	0	
	Very slow	0	
	Extremely slow	8	
Groundwater	Artesian	0	Suitable for all groundwater conditions but groundwater in the drillholes may affect the grout-ground bond in some rock types, especially shales and mudrocks; ”artesian” not applicable to the type of situation treated by rock bolting.
	High	6	
	Low	8	
	Absent	8	
Surface water	Rain	8	Not practical within or close to water courses.
	Snowmelt	8	
	Localized	6	
	Stream	0	
	Torrent	0	
	River	0	
Maturity		8	Well established technique, widely used where applicable. Often insufficent attention paid to durability.
Reliability		8	Relatively simple schematization and analysis. Possible pitfalls in the systematic identification of wedges or slabs to be treated.
Implementation		6	Requires access on steep slopes, implying specialist equipment. Works must be planned carefully to avoid exposing workers to rockfall from above.
Typical Cost		6	Typically moderate; access conditions may have a strong impact on cost.

Note
Ratings are given on a scale of 1 to 10; the higher the grade, the most suitable is the specific method under consideration to use in landslides of the given characteristics, evaluated individually. Overall suitability to specific case under consideration may be obtained by a weighted average of these ratings, with user defined weights. Zero rating means ”not applicable”

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6.7 ROCK BOLTING

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MITIGATION THROUGH REDUCTION OF HAZARD

6 TRANSFER OF LOADS TO MORE COMPETENT STRATA

6.8 STRAND ANCHORS

Description

Strand anchors are structural elements installed and grouted in predrilled holes in soil or rock to transmit an applied tensile load into the ground. They are typically manufactured from high strength low relaxation class 1860 MPa steel in strands 15.7 mm (0.6”) in diameter; the number of strands in ground anchors typically varies from 3 to 8 (Figure 1). Strand is typically the most economical tendon and often the most versatile due to its flexibility. The maximum length is nominally unlimited, since the strand can be manufactured and assembled in any length and it can be transported coiled; in practice, however, the maximum length is limited by drilling. Typical overall lengths are up to 35 – 40 m.

The basic components of a grouted ground anchor include the: (1) anchorage; (2) free stressing (unbonded) length; and (3) foundation or bond length. (Sabatini et al., 1999)

The anchorage is the combined system of anchor head, bearing plate and trumpet which allows a correct housing of the strands and of the wedge system and the transmission of the prestressing force from the strands to the ground surface or the supported structure; in permanent application it is provided also with a protection cap.

The free length represents the part of the anchor between the foundation or bond length and the head, in which the strands are free to elongate elastically during tensioning operations and transfer the resisting force from the bond length to the structure, nominally without load transfer to the surrounding ground. A bondbreaker is a smooth plastic sleeve filled with greese that is placed over the tendon in the unbonded length to prevent the prestressing steel from bonding to the surrounding grout. It enables the prestressing steel in the unbonded length to elongate without obstruction during testing and stressing and leaves the prestressing steel unbonded after lock-off, providing corrosion protection at the same time.

The foundation or bond length is the part of the anchor which transmits the tensile stresses to the ground; this is usually obtained by means of cement grout providing adherence between the tendon and the drillhole. To increase the grout-steel adherence, strands are suitably shaped by means of spacers and straps. A typical assembly is shown in Figure 2.

Appropriate drilling methods must be selected to suit ground and groundwater conditions, supporting the drillhole with casing, if required. Bentonite or other mud suspensions should not be used, as “smear” on the drillhole walls can significantly reduce the grout-to-ground bond. Air flush should be used in argillaceous soils and rocks susceptible to rempulding. Typical drillhole size range from 100 to 200 mm; a minimum inclination of 10° below horizontal is recommended to facilitate grouting. Typically, drillholes in rock are self supporting. However, critical drilling conditions with potential loss of borehole stability may be encountered when drilling through higly fractured or milonitic zones, especially if water is also encountered in the drillhole. In this case, it may be simpler to grout and redrill the hole, rather than using a casing.

Strand anchors contribute to the stabilization of ground slopes operating according to the scheme reported in Figure 1 of fact sheet 6.0 on the “General aspects of hazard reduction by transfer of loads to more competent strata” or in combination with other structures such as piles (fact-sheet 6.2), barrettes (fact-sheet 6.3) or caissons (fact-sheet 6.4).

Permanent anchors must satisfy three basic requirements:

5. There must be a suitable method of anchoring the distal end of the strands (foundation, bond lenth) in the drill hole;
6. A known tension must be applied to the strand anchor without creep and loss of load over time;
7. The complete strand anchor assembly must be protected from corrosion for the design life of the project.

The most common method of anchoring the distal end of a strand anchor in the drill hole is cement grout. There are several techniques to form the grouted bond length, as follows (Wymer et al., 2003):

A. Gravity grouted shaft borehole, which may be lined or unlined depending on hole stability;
B. Low pressure (< 1 MPa) grouted borehole via a lining tube or insitu packer where the diameter of the fixed anchor is increased with minimal disturbance as the grout permeates through the pores or natural fissures in the ground;
C. High pressure (> 2 MPa) grouted borehole via lining tube or insitu packer, where the grouted fixed anchor is enlarged via hydrofracturing or compaction of the ground;
D. Gravity grouted borehole in which a series of enlargements (underreams) have previously been mechanically formed.

In soils it is important to increase friction between grouting and the surrounding soil; therefore grouting of the bond zone is usually Type C, performed by means of pipes with valves equipped with manchettes placed at variable distance (typically 30 to 100 cm) depending on the soil characteristics, to permit repeated localized high pressure grouting and to repeat grouting after tensioning, if necessary, in case of insufficient friction.

Figure 1: Cut away view of typical multistrand tendon (source: Sabatini et al., 1999)

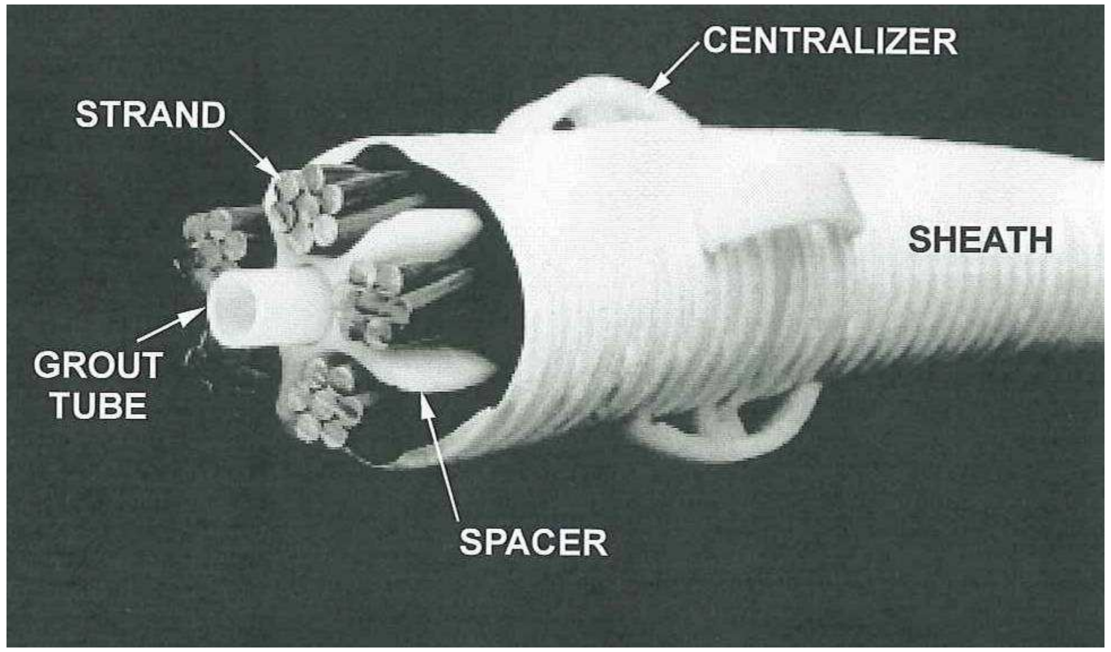
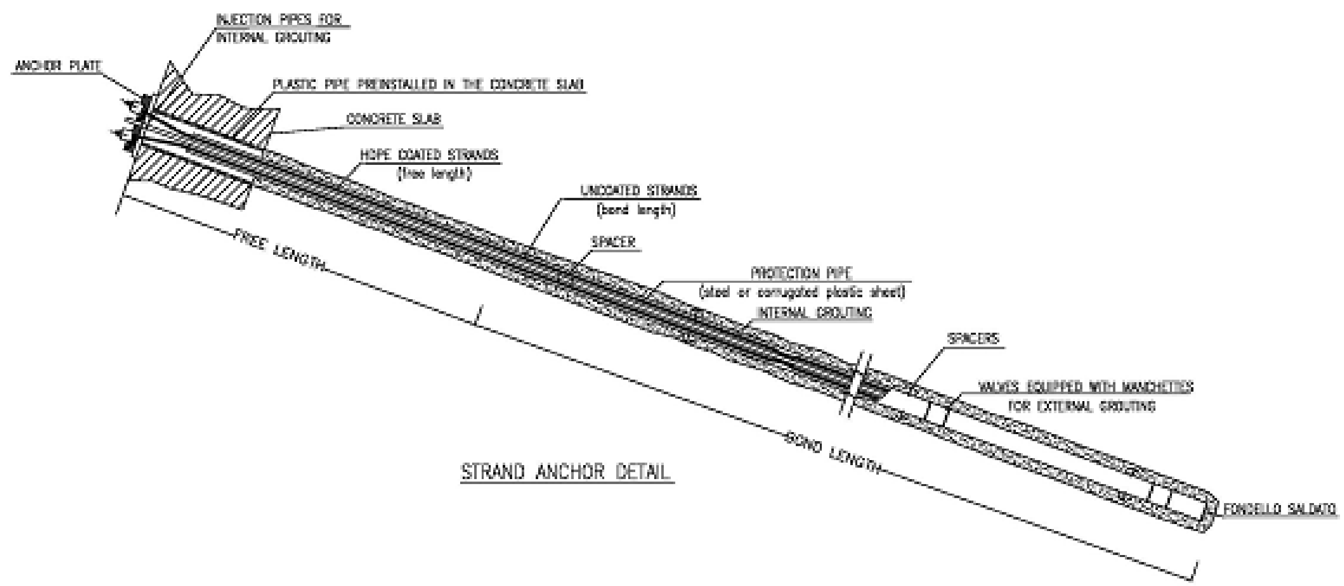


Figure 2: Multistrand anchor, typical detail (source: SGI-MI project files)



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6 TRANSFER OF LOADS TO MORE COMPETENT STRATA

6.8 STRAND ANCHORS

The most appropriate method to ensure that bolts are not susceptible to creep and loss of load over time is to set operating loads significantly lower than the pullout resistance and below the level at which significant creep or fluage is observed in load tests. Specific test procedures have been developed for example by the Post Tensioning Institute (1985) and by AICAP (1993), which can detect the essential aspects of the behaviour of the anchor and the surrounding ground, to determine also the long term pullout resistancet rather than the short term resistance only.

As experience with ground anchors in general and with strand anchors in particular accumulated, increasing attention has been focused on durability and corrosion protection. Ever since the publication of BS 8081:1989, all standards and guidelines place great attention on this issue.

The protection degree of strand anchors is defined with reference to their design life and to the environmental aggressiveness. Protection in every part of the strand anchor is usually assured by:

- Bond length: cement grouting and plastic, dielectric, waterproof and corrugated sheath (permanent anchors).
- Free length: each strand is coated with soft corrosion protection compounds (grease, wax, etc.) and contained in a polyethylene pipe; an external sheath covers the whole bundle of strands (temporary and permanent applications).
- Anchorage: it is the critical element of the system and is the part most susceptible to corrosion and to transmission of stray currents; it requires a perfect sealing above and below the bearing plate. <u>Protection below the bearing plate</u> is assured by an insulating system consisting of a cylindrical chamber sealed to the anchor head and to the plastic sheath in the free length; after tensioning the cylindrical chamber is filled with anticorrosion compound. <u>Protection above the bearing plate</u> is assured by a concrete sealing, if it does not require in service checking or re-tensioning or by the installation of a metallic cap filled with anticorrosion compound. Further protection from stray currents may be allowed by the interposition between bearing plate and anchor head of dielectric materials.

The current European standard (EN 1537:2000) classifies anchors depending on their design life, distinguishing ”temporary” and ”permanent” anchors, having a design life less than and more than 2 years respectively. Double corrosion protection and dielectric isolation are mandatory for permanent anchors and according to strict interpretation, in situ grout cannot be considered as providing corrosion protection.

Strand anchors are used in all types of soils and highly weathered and/or fractured rocks or where rock bolting is impractical due to the lengths required (overall length more than 15 m). Examples are shown in the schematic section of Figure 3 and in Pictures 1 to 3.

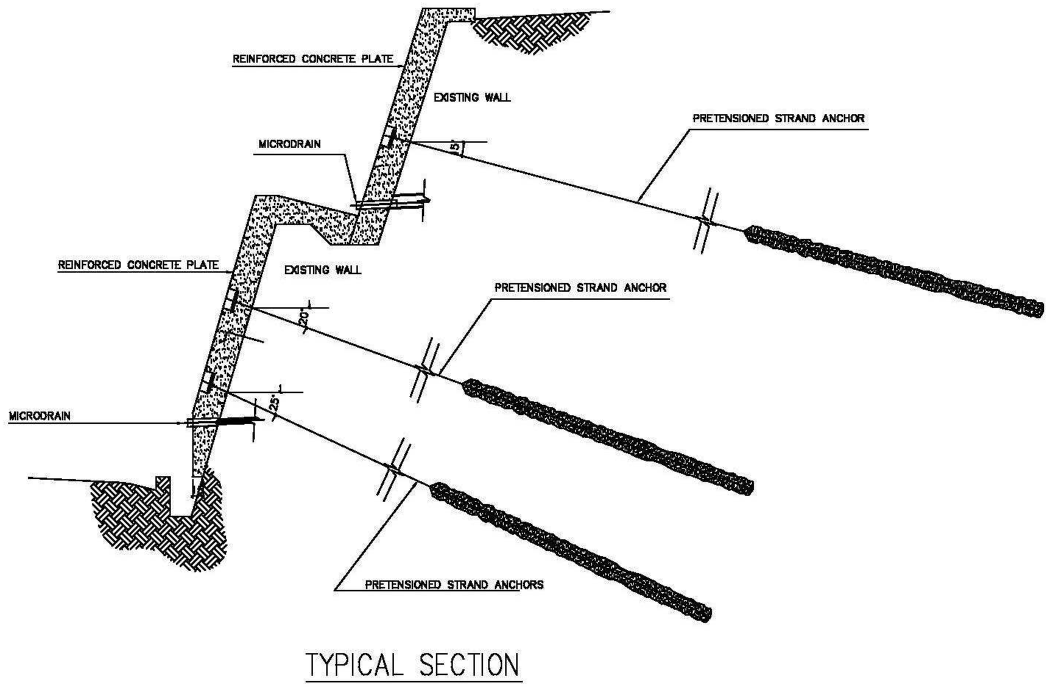
A load distribution structure is normally required to spread the very high concentrated loads available at the anchorage. Reinforced concrete spreader slabs or beams are normally used for this purpose in permanent applications. Where the soil or the weathered rock may degrade and ravel from under and in between the reinforced concrete slabs or beams, a full containment facing must be included in the anchorage system. Different solutions may be foreseen for the facing, including for example:

- Reinforced concrete walls: the wall acts both as a protection against raveling of the rock and as a large reaction plate for the rock bolts; the rock bolt will be drilled through sleeves in the concrete; it is also important that there be drain holes through the concrete to prevent buildup of water behind the wall.
- Shotcrete, reinforced with reinforcing mesh (typically steel, but other materials may be equally suitable).
- Reinforced wire mesh, with a network of steel cables.
- Reinforced wire mesh associated with reinstatement of vegetation.

A typical facing that was popular in the 1970’s and 1980’s is a network of “vertical” and “horizontal” reinforced concrete beams forming a grid pattern on the slope, with strand anchors at the intersections; the open spaces between the beams were typically filled with soil to encourage re-naturalization.

A description of the use of anchors to stabilizate landslide is provided by Millet et al. (1992); interesting case histories describing the use of tiebacks together with drilled shaft or driven H-piles walls are presented in Weatherby and Nicholson (1982), Hovland and Willoughby (1982), Tysinger (1982).

Figure 3: Schematic section of typical strand anchors application for slope stabilization (source: SGI-MI project files)



Picture 1: Strand anchors and concrete slab to stabilize multiple tier retaining wall (source: SGI-MI project files)

MITIGATION THROUGH REDUCTION OF HAZARD

6 TRANSFER OF LOADS TO MORE COMPETENT STRATA

6.8 STRAND ANCHORS

Design

For cases where strand anchors are used in conjunction with other mitigation measures such as piles, barrettes and caissons, they are explicitly considered in the respective global stability and soil-structure interaction analysis appropriate for each type of mitigation measure(see fact sheets 6.2, 6.3 e 6.4).

For stand alone anchored plates, the geotechnical design is carried out using the methods and criteria set out in fact-sheet 2.0 on the "general aspects of mitigation through changes in slope geometry and/or load distribution), taking into account the stabilizing effect of the anchor loads. Besides global stability analysis, it is necessary to verify the bearing capacity of isolated spreader slabs or beams, the local stability between isolated spreader slabs or beams, the adequacy of any facing and the structural design of slabs, beams and walls.

With regard to the design of the bond length, it should be noted that the stress distribution along the bond length is higly non-uniform. In practice, the required length of the bond zone can be calculated with the simplifying assumption that the shear stresses at the ground-grout interface is uniformly distributed along the bond length. In this respect, it is recommended to restrict the length of the foundation to maximum 12 to 15 m.

Limit values of the shear stresses at the grout-ground interface can be estimated by applying, for example, the recommendations given by Bustamante and Doix (1985) which consider different values of limiting friction as a function of both ground characteristics and the method of grouting. Care should be excercised in applying these or similar guidelines to anchors placed at shallow depth where the limited cover may preclude or render ineffective high pressure grouting.

The working shear strength of the corrugated sheat-grout interface is usually greater than the working strength of the ground-grout interface; for this reason the length of the bond zone is typically determined from the stress level of the ground-grout interface.

In all cases, it is highly recommended to verify the actual limit resistance of the anchor by full scale preliminary load tests before starting commercial production. A suitable testing procedure to check that the full design load is applied at the required depth and that there will be no loss of load with time shall be drawn; reference can be maid to recommendations given, for example, by the Post Tensioning Institute (1985) and by the AICAP (1993).

As far as experiences on maintenance and monitoring of permanent anchors are concerned, reference can be made to the paper by Littlejohn and Mothersille (2008a; 2008b).

Picture 2: Anchorage for 4 strand anchors; photograph shows an example of poor quality construction, both with respect to concrete quality and to prevention of corrosion (source: SGI-MI project files)

Picture 3: Strand anchors and concrete slab for slope stabilization (source: SGI-MI project files)

MITIGATION THROUGH REDUCTION OF HAZARD

6 TRANSFER OF LOADS TO MORE COMPETENT STRATA

6.8 STRAND ANCHORS

APPLICABILITY

Class	Descriptor	Rating	Notes
Type of movement (Cruden & Varnes, 1996)	Falls	6	Suitable for a wide variety of situations, provided they can reach stable ground; cannot prevent or contrast spreads and flows.
	Topples	8	
	Slides	8	
	Spreads	0	
	Flows	0	
Material	Earth	8	Suitable in all types of materials, some difficulty may be encountered drilling in debris, whcih may give rise to both relatively hard drilling (especially if loose hard blocks are encountrered), problems with drillhole stability and water inflow.
	Debris	6	
	Rock	8	
Depth of movement	Superficial (< 0.5 m)	0	Most suitable for deep and very deep movemenets requiring very long anchors, where the advantadges of strand over bars are evident; often used also in medium depth movements in association with piles.
	Shallow (0.5 to 3 m)	2	
	Medium (3 to 8 m)	6	
	Deep (8 to 15 m)	8	
	Very deep (> 15 m)	8	
Rate of movement (Varnes, 1978)	Moderately to fast	0	Movement must be extremely or very slow to allow installation. Drilling and placing the anchors typically takes several hours but grouting operations typically take several days.
	Slow	0	
	Very slow	4	
	Extremely slow	8	
Groundwater	Artesian	2	Difficulties may encountered with drilling and grouting where groundwater levels are high or, worse, artesian. Forming drillholes with the mouth below the groundwater table in relatively free draining soils can be problematic or even not feasible.
	High	6	
	Low	8	
	Absent	8	
Surface water	Rain	8	Water courses need to be diverted to allow installation; the presence of water courses may accelerate corrosion and prevent inspection and maintenance.
	Snowmelt	8	
	Localized	8	
	Stream	2	
	Torrent	0	
	River	0	
Maturity		6	There is scope for further development, especially in terms of manufacturing technology, corrosion potection system and the use of new materials, (FRPs).
Reliability		6	Critical item, where used. Reliability, especially in the long term. depends on correct detailing and installation procedure; doubts on long term durability.
Implementation		6	Can be implemented with commonly available equipment but requires special expertise.
Typical Cost		6	Moderate, in relation to the benefits provided, so long that installation does not require special access provisions.

Note
Ratings are given on a scale of 1 to 10; the higher the grade, the most suitable is the specific method under consideration to use in landslides of the given characteristics, evaluated individually. Overall suitability to specific case under consideration may be obtained by a weighted average of these ratings, with user defined weights. Zero rating means "not applicable"

MITIGATION THROUGH REDUCTION OF HAZARD

6 TRANSFER OF LOADS TO MORE COMPETENT STRATA

6.8 STRAND ANCHORS

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AICAP (1993). ”Ancoraggi nei terreni e nelle rocce – Raccomandazioni”. In Italian, 43pp..

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Bustamente M., Doix B. (1985). ”Une méthode pour le calcul des tirants et des micropieux injectés”. Bull. Liaison labo P. Et Ch., 140, ref. 3047, 75-92.

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Hovland H.J., Willoughby D.F. (1982). ”Slide stabilization at the Geyser’s Power Plant”. In Application of Walls to Landslide Control Problems, Proc. of two sessions, ASCE National Convention, Las Vegas, Reeves R.B. editor, 77-92.

Littlejohn, S. (1992). "Ground Anchorage Technology - A Forward Look." Proceedings of the Conference on Grouting, Soil Improvement and GeoSynthetics, Vol. 1, Geotechnical Special Publication No. 30, ASCE, New Orleans, Louisiana, pp. 39-62.

Millet R.A., Lawton G.M., Repetto P.C., Varga V.K. (1992). ”Stabilization of Tablachaca Dam”. Proc. of a Specialty Conference on Stability and Performance of Slopes and Embankments, Berkeley, California, R.B. Seed and Boulanger R.W. editors, Geotechnical Special Publication 31, ASCE,.1365-1381.

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MITIGATION THROUGH REDUCTION OF HAZARD

FACT SHEET 7

RETAINING STRUCTURES

MITIGATION THROUGH REDUCTION OF HAZARD

7 RETAINING STRUCTURES (TO MODIFY SLOPE GEOMETRY AND/OR TO TRANSFER LOADS TO COMPETENT LAYER)

7.0 GENERAL

Description

The potential benefits and methods of modifying the geometry and/or mass distribution of slopes not susceptible to flow slides are described in Section 2 of Annex A; where feasible, loading at the toe with buttress fills is shown to be particularly effective and relatively inexpensive. Where buttress fills at the toe of the slope are not feasible due to geometrical or other constraints, such as the presence of existing structures, infrastructures, etc., retaining structures may provide a workable solution, limiting the space occupied by the stabilizing mass. In certain conditions reinforced soil structures may be used also to replace the unstable mass altogether.

Depending on the specific conditions and constraints at the site, retaining structures may either rest on top of or within the sliding mass (Figure 1), whereby their stabilizing effect depends entirely on the modification to the mass distribution on the slope, or they can penetrate through the entire thickness of the sliding mass, reaching and thus transferring loads to more competent layers below (Figure 2). Structures which are designed primarily with this latter purpose are described in detail in Section 6 of Annex A. In this Section, the attention is focused on structures where the transfer of loads to more competent layers, where it occurs at all, is only part of the stabilizing mechanism.

Depending on performance requirements, availability and durability of materials, local practice, aesthetics, cost and speed of construction, retaining structures can be of different types, as follows (Chapman et al., 2000):

- Reinforced soil structures;
- Gabion walls;
- Crib walls
- Drystack masonry walls
- Mass concrete or masonry walls
- Reinforced concrete stem walls (cast in situ or prefabricated)

As a general rule, for slope stabilization relatively flexible retaining structures should be preferred to rigid structures, since the latter are more susceptible to being damage by differential movements.

Modular walls, such as prefabricated RC stem walls, crib walls and gabion walls are typically relatively fast to construct and factory made structural elements are more amenable to systematic quality control before they are incorporated into the wall.

Systems such as crib walls, gabion walls and the various types of reinforced soil systems, which combine speed of construction with relative flexibility, are increasingly more common.

The use of certain systems may be inappropriate or may require special precautions where the structure may be subjected to vandalism or accidental mechanical damage, for example by vehicles or fire.

In all cases, special care needs to be paid to ensure that the structure does not impede the correct drainage of groundwater.

Design

The basic design of the retaining structures is similar to the design of buttress fills; under the thrust exerted by the sloping ground (unstable or potentially unstable) the retaining structures shall be verified, with adequate factors of safety, against external instabilities such as:

- Overturning;
- Sliding at or below its base;
- Bearing failure of the foundation.
- Overall stability, both locally to the structure and including the unstable or potentially unstable ground behind.

The internal stability of reinforced concrete walls is assured by conventional structural checks; all other types of retaining structures require the internal stability to be verified by appropriate methods, including separate structural and geotechnical verification of facing and reinforcing elements, where appropriate.

References:

Chapman T., Taylor H., Nicholson D. (2000). “Modular Gravity Retaining Walls – Design Guidance”. Publication C516, CIRIA, London

Figure 1

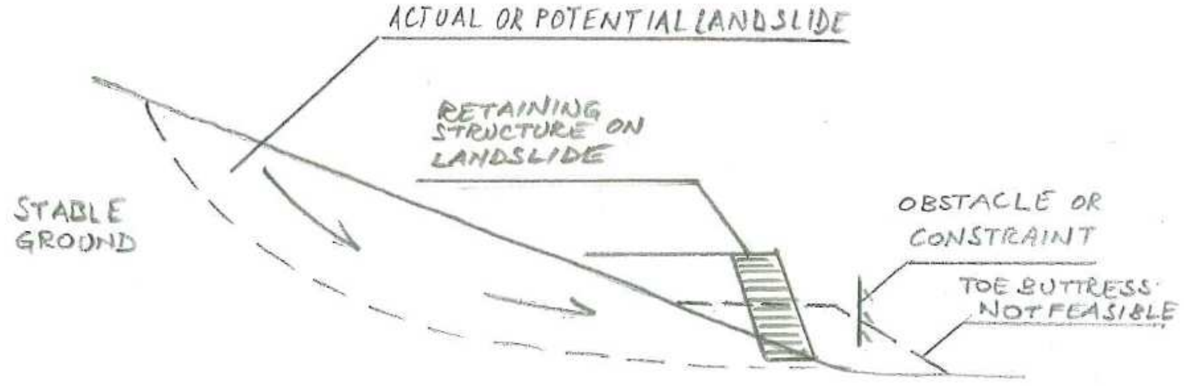
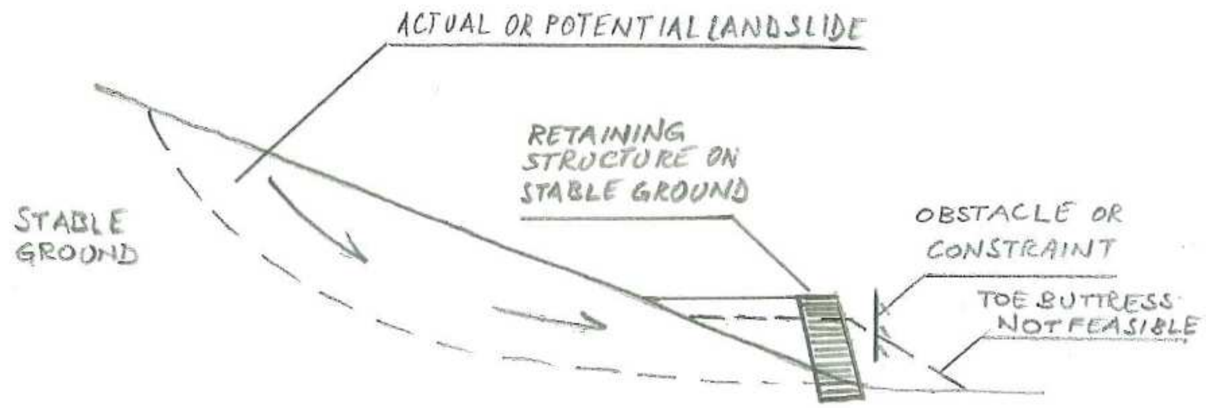


Figure 2



MITIGATION THROUGH REDUCTION OF HAZARD

7 TRANSFER OF LOADS TO MORE COMPETENT STRATA

7.1 REINFORCED SOIL STRUCTURES

Description

Reinforced soil structures (Figure 1) are formed by compacted layers of soil 50 to 150 cm thick in which reinforcing elements of appropriate length are interposed to improve overall resistance; the external face of the structure is protected by a facing which may consist of shotcrete and wire mesh, geogrid/geotextile sheets, modular facing blocks, cast-in-situ or prefabricated panels or similar (Figure 2). The facing may incorporate biotechnical elements, typically for aesthetic purposes only.

Reinforced soil structures are generally applicable to situations where the reinforcement elements and the fill are placed as the wall is constructed. The concept of reinforcing the backfill behind retaining walls was developed by H. Vidal in France in the mid 1960s).

These structures offer several advantages As highlighted for example by Mitchell and Villet (1987), reinforced soil structures:

- are coherent and flexible to tolerate relatively large displacements;
- can use a wide range of backfill materials;
- are easy to construct;
- are relatively resistant to seismic loading; however their use in areas of high seismicity is still somewhat restricted because of the lack of definitive research on this issue; in particular the connection between the reinforcing elements and the facing elements may be critical (Allen and Holtz, 1991).
- can form aesthetically attractive retaining walls and slopes because of available facing types
- are often less costly than conventional retaining structures, especially for high steep slopes and high walls.

General principles

In reinforced soil structures the reinforcing elements provide the structure with a component of tensile strength. As the height of the wall increases, the overburden pressure increases and the shear stresses within the soil mass build up. There is a tendency for the face of the wall to displace outwards which increases as the height of the wall increases. The outward movement of the soil is resisted by the reinforcing elements which go into tension as frictional forces develop along them. Because of the thin nature of the reinforcing elements used in this type of structure, they can only provide tensile resistance. The tensile forces acting in the reinforcements also contribute to the normal stress acting along potential slip-surfaces within the reinforced soil mass, thus increasing the frictional resisting force along them. In the case of reinforcements consisting of grid mesh, with orthogonal strips running parallel to the face of the wall, there is also a component of resistance generated from their edge bearing against the soil infilling the gaps between the strips.

The maximum tensile forces in the elements occur within the reinforced soil mass rather than at the facing. The locus of the point of maximum tensile force in each row of reinforcing elements separates the reinforced soil mass in two distinct zones, an “active” zone immediately behind the facing and a “passive” zone. Contrary to soil-nailing structures, the position of the line of maximum tension can be reasonably estimated in cases of reinforced soil structures due to their uniform geometry and the “known” characteristics of materials.

Reinforcing elements

The reinforcing elements may consist of:

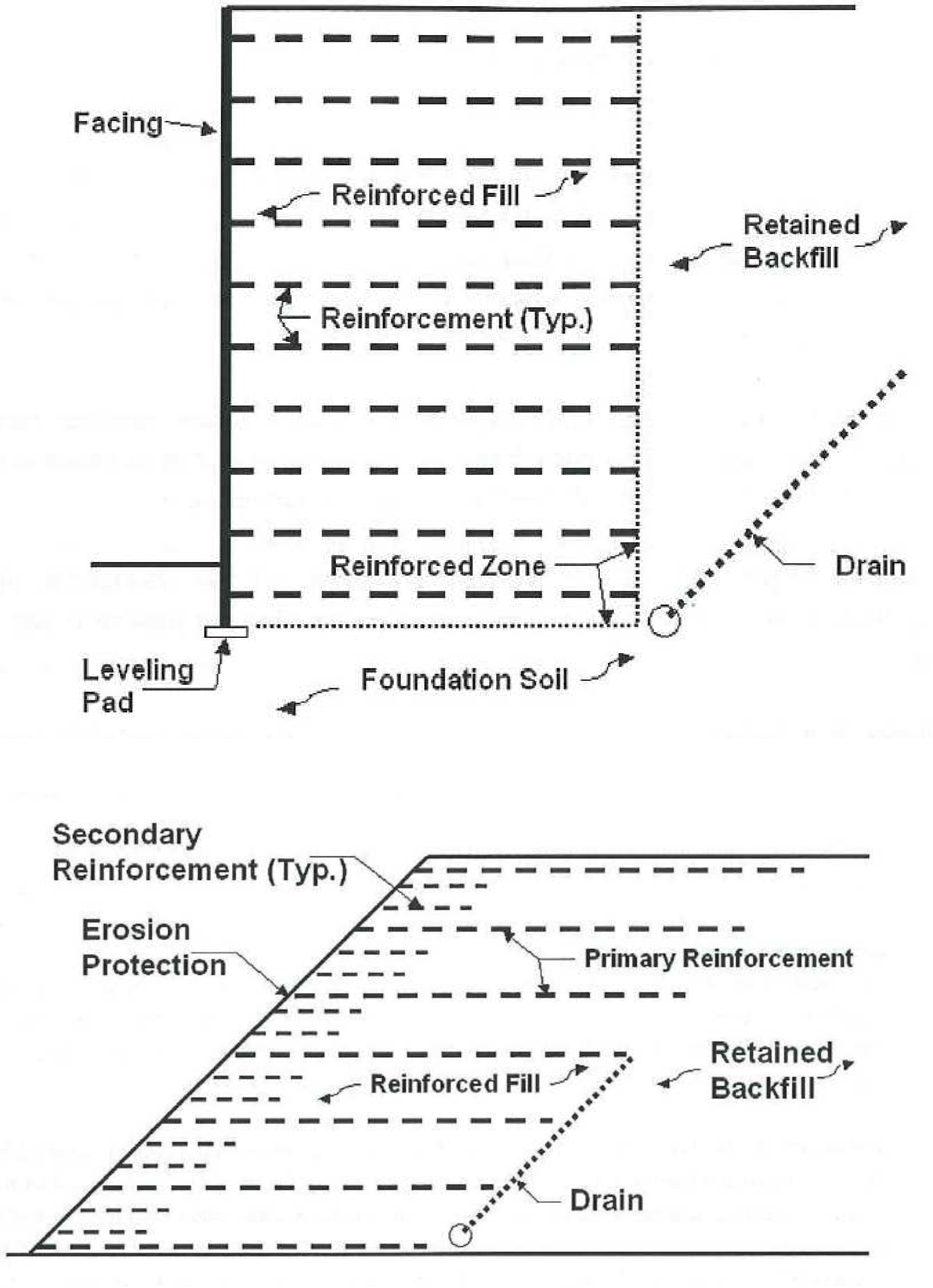
- Metallic strips (Reinforced Earth or Terre Armée);
- Polymeric strips;
- Geotextile sheets;
- Geogrids;
- Metallic grids.

<u>Strip reinforcing elements</u>

The mechanism of stress transfer between the reinforcement and the soil is essentially friction developed at the surface of the reinforcing strip (Mitchell and Villet, 1987; Christopher and Holtz, 1989; Christopher et al., 1990).

Early experiments with fibreglass-reinforced polymers, stainless steel and aluminium strips were not successful so all Reinforced Earth (Terre Armée) walls are currently constructed using galvanized steel strips (Schlosser, 1990).

Figure 1: Generic cross section of reinforced soil walls and slopes (source Berg et al., 2009)



MITIGATION THROUGH REDUCTION OF HAZARD

7 TRANSFER OF LOADS TO MORE COMPETENT STRATA

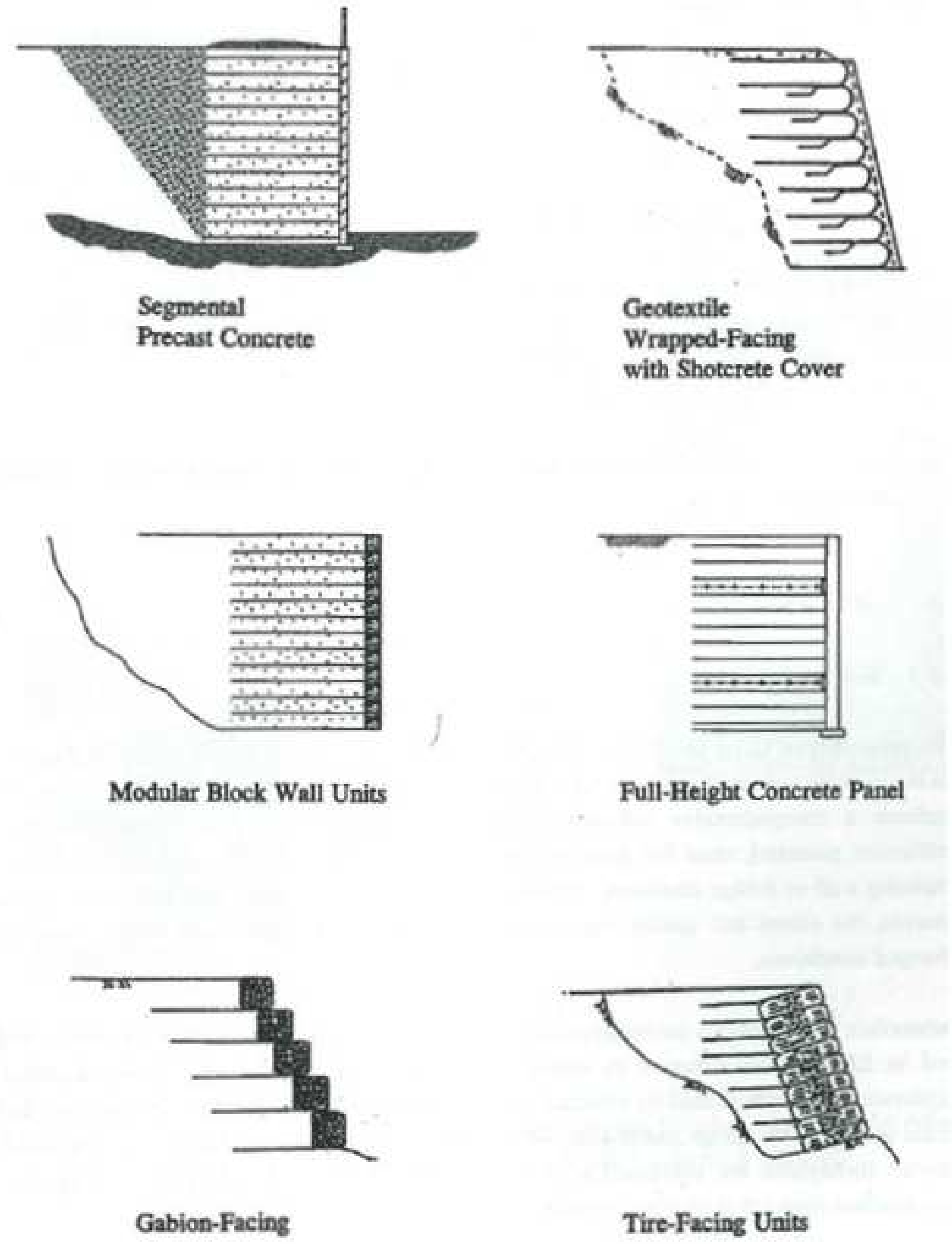
7.1 REINFORCED SOIL STRUCTURES

As corrosion rates of metals in soil are very difficult to predict, also in presence of galvanized steel strips free-draining sand and gravel fills are specified to reduce corrosion potential. Epoxy-coated steel strips have been developed and may offer higher resistance to corrosion (Elias, 1990). In theory, steel reinforcement could be designed with a sacrificial thickness, but this is seldom economic considering the small initial thickness of the reinforcement elements and the need to provide sacrificial steel all round.
Since the mid 1970s, non-metallic strips have been also developed (Holtz, 1978; Jones, 1978), consisting of continuous glass fibres embedded in a protective coating of epoxy resin or of geosynthetic strips.
The reinforcement elements are connected to vertical prefabricated reinforced concrete panels or inclined steel mesh facing panels progressively assembled as the structure is constructed.
In an attempt to improve the stiffness and pull out resistance of the reinforcement, bar-and-mesh systems or bar-mats formed by cross-linking steel reinforcing bars were developed by California Department of Transportation, Caltrans (Forsyth, 1978); laboratory tests showed that the bar-and-mesh reinforcement could produce significantly higher pull-out resistances compared to longitudinal bars only (Chang et al., 1977). Evolving from the Caltrans project other bar mats systems has been developed and used (see for example Anderson et al., 1987; Hausmann, 1990; Mitchell and Christopher, 1990). The main problems with bar mats systems are given by the corrosion of the steel bars.

Getextile sheets
The use of geotextiles in reinforced soil structures followed shortly after the introduction of Reinforced Earth (Terre Arméè), (Bell and Steward, 1977; Yako and Christopher, 1988; Allen at al., 1992).
The mechanism of stress transfer between the reinforcement and the soil is essentially friction developed at the surface of the reinforcing sheets (Mitchell and Villet, 1987; Christopher and Holtz, 1989; Christopher et al., 1990).
A large variety of nonwoven or woven polyester and polypropylene geotextiles, with a wide range of mechanical properties, is available (Christopher and Holtz, 1989; Koerner, 1990).
Coarse grained soils ranging from silty sands to gravels are commonly used as fill.
The most common facings are formed by wrapping the geotextiles around the exposed soil. Since the geotextiles are subjected to vandalism, mechanical damage and deterioration, the exposed materials must be covered with shotcrete or asphalt emulsion, modular facing elements, gabions or soil and vegetation. In the latter case, the facing typically includes additional layers specifically designed to control erosion, consisting of variable combinations of geogrids, geomats and/or biodegradable mats, to hold the soil in place until the vegetation has taken hold.
The use of geosynthetics sheets instead of steel strips has been introduced and it has become progressively more popular mainly on account of their lower cost and greater corrosion resistance. However, doubts persist on the durability and longevity of geosynthetic materials because of chemical and biological attack (Elias, 1990; Allen, 1991; Brand and Pang, 1991). The mechanical characteristics of geosynthetics also give rise to issues related to their lower stiffness and their susceptibility to significant creep (Rimoldi and Ricciuti, 1992).

Geogrids and Metallic grids
In grid reinforcement, polymeric or metallic elements are arranged in rectangular grid shape, with the long side oriented parallel to the direction of the movement between the reinforcement and the soil; therefore, the grid-soil interaction involves both friction acting on the long side grid elements and passive bearing resistance on the short side grid elements. Due to the contribution of the passive bearing resistance grid reinforcements provide higher resistances to pull-out than flat strips; it should be considered, however, that passive bearing resistance develops after relatively large displacements (5 to 10 cm), see for example Schlosser (1990).
Polymeric geogrids represent the most commonly used element for soil reinforcement; they are made by polypropylene, polyethylene or PVC coated polyester. Since the 1970s, advances in the formulation of polymers led to significant improvement in their strength and stiffness and in their use for several applications, including repair of slope failures (O'Rourke and Jones, 1990; Murray and Irwin, 1981; Murray, 1982; Jones, 1985; Szymoniak et al., 1984; Forsyth and Bieber, 1984; Mitchell and Christopher, 1990). As with the geotextile sheets, polymeric geogrids are susceptible to environmental deterioration, large deformations and creep.
Coarse grained soils ranging from silty sands to gravels are commonly used as fill.
Requirements and details of facings are similar to those described above for structures constructed with geotextile sheets.

Figure 2: Types of reinforced soil wall facing (after Wu, 1994, as reported by Berg et al., 2009)



MITIGATION THROUGH REDUCTION OF HAZARD

7 TRANSFER OF LOADS TO MORE COMPETENT STRATA

7.1 REINFORCED SOIL STRUCTURES

Whatever the reinforced soil structures, provision of drainage behind the facing and the reinforced soil mass is important, to maximize effective stresses within the fill and available shear strength at the soil reinforcement interfaces. Allowance for drainage from the facing should also be made.

For a more comprehensive description and discussion on reinforced soil structures reference can be made, for example, to Lee et al. (1973), Jones (1985), Mitchell and Villet (1987), Christopher et al. (1990), Mitchell and Christopher (1990), O'Rourke and Jones (1990), DoT Advice note HA/68/94 (1994), BS 8006 (1995), Love and Milligan (1995), Jewell (1996), Jones (1996), Berg et al. (2009).

Design

The thickness of the layers used depends on the nature of the fill and reinforcement and on the geometry of the structure.

The filling material shall be suitable for compaction; granular fill is typically compacted to 95% of the maximum dry density determined in Modified Proctor Test.

The type of reinforcement, facing and connections depend on soil type, wall height, slope, etc. Usually polymeric geosynthetic are considered extensible, while steel strips are considered inextensible; using extensible or inextensible reinforcements may determine differences in the method of analysis (see for example BS 8006, 1995).

The toe of the facing should be embedded below the ground surface to prevent against local punching failure at the base of the facing and to prevent flow of the soil under the wall from water flow due to a head building up behind the facing (piping).

The basic design of the reinforced soil structures under the thrust exerted by the (unstable or potentially unstable) sloping ground behind it includes both external and internal stability evaluations (see for example Ghionna, 1995).

External stability evaluation will include consideration of:

- Overturning;
- Sliding at or below the base;
- Bearing failure of the foundation.
- Overall stability including the unstable or potentially unstable ground behind the reinforced soil system.

They are carried out in static and seismic conditions according to simplified methods normally adopted for conventional earth retaining structures.

Sliding should be checked using the weakest relevant frictional properties considering that sliding might occur through the foundation soil, the fill or along the interface of the reinforcing element used at the base of the structure.

Although not a stability criterion, the settlement of the ground induced by the reinforced soil system should be considered; excessive settlements can cause problems for example with drains and services; care should be taken when earth retaining structures are built adjacent to other existing structures.

Internal settlements of reinforced soil structures are governed by the nature and compaction of the fill and the vertical stresses within it (which depend on the height of the structure and surcharges). Differential settlements generally cause the most severe effects on a completed structure; the facing is the most critical part of the structure.

The internal stability is checked for each stage of construction to ensure that failure does not occur in/or around the reinforcements and the facing.

The internal stability is checked by limit equilibrium methods in which the additional forces provided by the reinforcing elements are added. Only ultimate limit states are examined with these methods; the magnitude of deformations, which govern the serviceability limit states, is usually controlled by applying adequate factors of safety to account for variations in material properties, loads, methods of analysis, etc. However, where deformations are critical it is necessary to resort to numerical analysis to estimate displacements.

The internal stability evaluations will include:

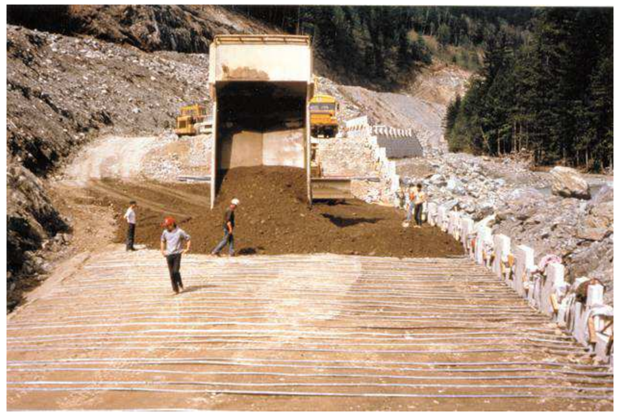
- Structural checks on the reinforcement, to verify that their tensile strength is sufficient to withstand with adequate factors of safety the tensile forces generated by the interaction with the soil.
- Geotechnical checks on the reinforcement, to verify that their length is sufficient to provide adequate pull-out resistance to withstand with adequate factors of safety the tension generated by the interaction with the soil.
- Structural checks on the facing, with particular reference to the connections with the reinforcement and local bending and shear in the facing

Figure 3: Typical construction sequence and detaild for reinforced soil wall with concrete panel facing (source: www.recocanada.ca, La Terre Armée Internationale® - The Reinforced Earth Group®)

Placing wall panels

Placing reinforcing strips, sheets or grids

Connecting reinforcment to wall panels



Backfilling to next layer of reinforcment

Figure 4: RSS with sheet reinforcement and soil bags facing (source: www.geosynthetycsmagazine.com)

Figure 5: RSS with grid reinforcement and concrete block facing (source: www.southeastrrsupply.com)

MITIGATION THROUGH REDUCTION OF HAZARD

7 TRANSFER OF LOADS TO MORE COMPETENT STRATA

7.1 REINFORCED SOIL STRUCTURES

Various methods of analysis have been developed to evaluate these three aspects; the various methods have been validated to different extent with full scale experiments and instrumentation. A comprehensive descriptions of the various methods can be found, for example in DoT Advice note HA/68/94 (1994), BS 8006 (1995), Love and Milligan (1995), Jewell (1996), Jones (1996), Berg et al. (2009).

To account for creep and temperature effects, the rupture strength of polymeric reinforcements is governed by the characteristic strength corresponding to the required design life and temperature conditions; partial factor of safety are applied to this strength to account for both variations in material strength and environmental conditions, possible damage during construction, the need to limit creep deformations and the extent to which extrapolation of experimental data is required where the design life of the structure exceeds available long term tests. The rupture strength of metallic strip reinforcements is usually the quoted yield strength of the material:

The bond resistance is usually determined from considerations of the frictional properties at the interface between the soil and the reinforcing elements, estimating normal stress acting at the interface. This approach is well suited to reinforced soil structures where the reinforcing material and fill are reasonably well controlled. The angle of friction is obtained from direct shear box tests with shearing carried out on the reinforcing materials at an appropriate range of normal stresses or from large scale pull out tests under controlled conditions.

Figure 6: RSS with grid reinforcement and gabion facing (source: officine maccaferri)

Figure 7: RSS with grid reinforcement and inclined facing (source: officine maccaferri)

Figure 8: RSS with grid reinforcement and gabion facing – before and after (source: officine maccaferri)

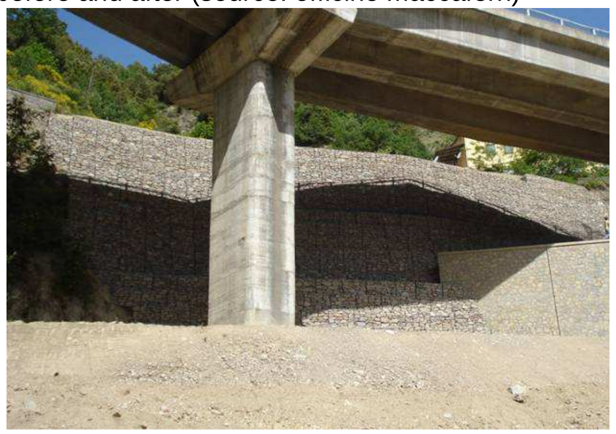
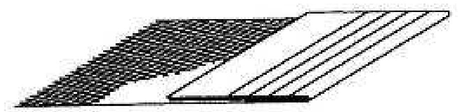
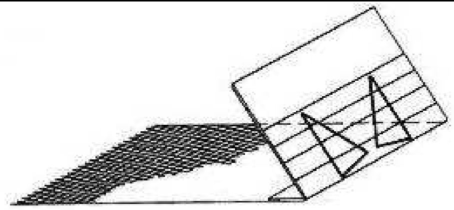
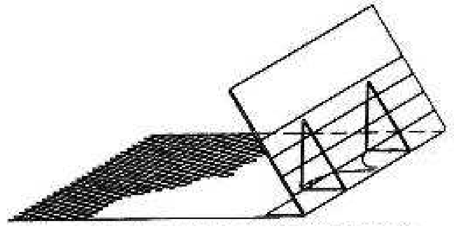
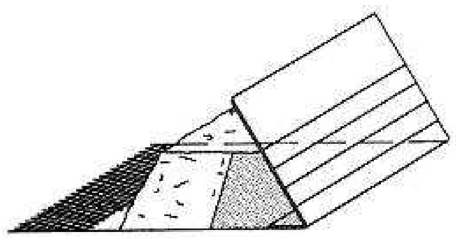


Figure 8: RSS with inclined facing arranged to support vegetation – typical assembly (source www.hydrogeo.net)

 Facing panels are supplied pre-folded, inclusive local reinforcment, geosynthetic, mesh facing and supports	
 facing panel is placed by hand and unfolded	
 temporary supports are rotated into position and fixed to the base reinforcment	
 Filling and compaction to the next level. Vegetable soil and light compaction near the facing. The excess reinforcment is folded back, and the sequence is repeated at the next layer	 

MITIGATION THROUGH REDUCTION OF HAZARD			
7 TRANSFER OF LOADS TO MORE COMPETENT STRATA			
7.1 REINFORCED SOIL STRUCTURES			
APPLICABILITY			
Class	**Descriptor**	**Rating**	**Notes**
Type of movement (Cruden & Varnes, 1996)	Falls	0	Most suited to rotational or pseudo-rotational slides. May be useful to reduce toppling hazard in certain conditions
	Topples	2	
	Slides	8	
	Spreads	0	
	Flows	0	
Material	Earth	8	Mainly applicable to landslides involving earth and debris. Applicability in rock limited by typical slope geometry and failure mode
	Debris	6	
	Rock	4	
Depth of movement	Superficial (< 0.5 m)	0	Typically applicable to intermedite depth landslides. Minimum size of reinforcment makes this approach impractical for very small or shallow landslides. Potentially the only suitable technique for very tall retaining structures, but the implications of large scale filling and procurement typically make it impractical for deep and very deep slides.
	Shallow (0.5 to 3 m)	4	
	Medium (3 to 8 m)	8	
	Deep (8 to 15 m)	6	
	Very deep (> 15 m)	2	
Rate of movement (Varnes, 1978)	Moderately to fast	0	Should be carried out preferably on very or extremely slow landslides; with due care it can be carried out in slow landslides
	Slow	4	
	Very slow	8	
	Extremely slow	8	
Groundwater	Artesian	8	Applicable in all groundwater conditions. Adequate drainage must be provided at the interface between low permeability fills and natural soil
	High	8	
	Low	8	
	Absent	8	
Surface water	Rain	6	Special facing detailing required where the structure is or can come in contact with flow. Mechanical damege of facing from solid transport typically precludes use near torrents.
	Snowmelt	6	
	Localized	6	
	Stream	4	
	Torrent	0	
	River	2	
Maturity		8	Relatively simple technique. Potential benefits and limits of applicability are well established.
Reliability		8	The reliability of the technique depends on the reliability of the evaluation of the stability of the treated slope and of the foundations.
Implementation		8	Downgrade to 6 where heavy modular elements need to be lifted using cranes in confined workplaces or on steep slopes
Typical Cost		6	Moderate to high, provided the work does not involve diversion of major water courses or interference with existing infrastructure.

Note
Ratings are given on a scale of 1 to 10; the higher the grade, the most suitable is the specific method under consideration to use in landslides of the given characteristics, evaluated individually. Overall suitability to specific case under consideration may be obtained by a weighted average of these ratings, with user defined weights. Zero rating means "not applicable"

MITIGATION THROUGH REDUCTION OF HAZARD

7 TRANSFER OF LOADS TO MORE COMPETENT STRATA

7.1 REINFORCED SOIL STRUCTURES

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MITIGATION THROUGH REDUCTION OF HAZARD

7 TRANSFER OF LOADS TO MORE COMPETENT STRATA

7.2 GABION WALLS

Description

Gabions are wire mesh boxes filled with stones, placed side by side and laced together in order to form a gravity structure; gabion walls can be built with either the front face or rear face stepped; where possible, it is desirable to incline the wall 6 to 8° from the vertical towards the backfill materials; typical schemes are shown in Figure 1; typical applications are shown in Figures 2 and 3.

Gabions are manufactured either from woven continuous wire or welded mesh; the steel mesh is protected against corrosion by a zinc or zinc-aluminium coating; in highly aggressive environments, additional bounded plastic, thermoplastic or epoxy resin polymer coating can be provided; further details on steel protection are provided by Chapman et al (2000).

The materials used to fill gabions must be durable, e.g. resistant to erosion and frost.

Gabion walls are constructed in 1 to 0.5 m high courses; gabions are typically supplied flat and assembled on site (Figures 4 and 5). Filling can be carried out mechanically (Figure 6) or by hand, depending on the finish required. In order to facilitate construction the backfill is placed and compacted keeping it to the same level reached by the wall. Gabions are suitable also for underwater uses; in this case, prefilled gabions are lowered and put in place by a crane, using a lifting frame; this construction process may be adopted also for other areas with poor access.

Gabion walls are permeable and will allow retained fill to drain freely; where appropriate or necessary, surface and/or deep drainage systems will be provided to keep the backfill materials free from groundwater pressures.

Gabion walls can be designed to support vegetation using growing pockets; root growth within or near the gabion structure is not normally detrimental. Advice on planting vegetation in gabion walls can be found in Coppin and Richards (1990). Care should be taken both in the choice of plants suitable for locations within, above or below the wall and for the suitability of the growing medium (usually loose topsoil or growbags) which may require special water retention measures.

Unprotected gabions are susceptible to vandalism, accidental damage and fire due to the small section size of the wire mesh.

Design

The wall specification should stipulate the materials to be considered for both gabion walls and for backfilling.

Stones for filling gabions should conform to BS 5390 for hardness, crushing strength and resistance to weathering (frost susceptibility in particular); they should conform also to specification provided in paragraph 8.1.2. of Chapman et al. (2000) regarding grain composition in relation also to the sizes of the gabion compartment and of the wire mesh.

The properties of backfill will depend on whether or not locally-won backfill is to be used, and if material is required to be free-draining. Optimum backfill is: easy to compact, giving high strength and stiffness; and free-draining, to minimize the build-up of groundwater pressure.

Backfill should not include: natural or contaminated soil which will be chemically aggressive; frozen materials; degradable materials such as topsoil, peat, wood, vegetation, etc.; materials which could be toxic, dangerous or prone to spontaneous combustion; soluble material or collapsible soils. The use of clays prone to swelling should be carefully considered as they can exert very high pressures on the back of retaining walls; the same applies for materials derived from argillaceous rocks such as shales and mudstones.

Walls design shall put special consideration on aspects related to water pressure and drainage. Rationale for drainage systems and related details can be found for example in Geotechnical Engineering Office (1993) and Chapman et al. (2000).

The following ultimate limit states (ULS) need to be verified:

- Bearing resistance failure at the base of the wall;
- Sliding failure at the base of the wall;
- Failure by toppling of the wall;
- Loss of overall stability around the wall;
- Overall stability of the slope, including the wall;
- Unacceptable leakage through or beneath the wall;
- Unacceptable transport of soil grains through or beneath the wall;
- Internal stability. Gabion walls shall be also proportioned so that the resultant forces at any horizontal section lies within the middle third of that section. No allowance should be made in the design for the strength of the wire. Analyses should be made on horizontal sections above the base of the wall to check that there is adequate resistance to sliding using a design friction angle for the gabion fill sliding against itself, ignoring the effect of the wire mesh.

Figure 1: Typical details of gabion walls (source Chapman et al., 2000)

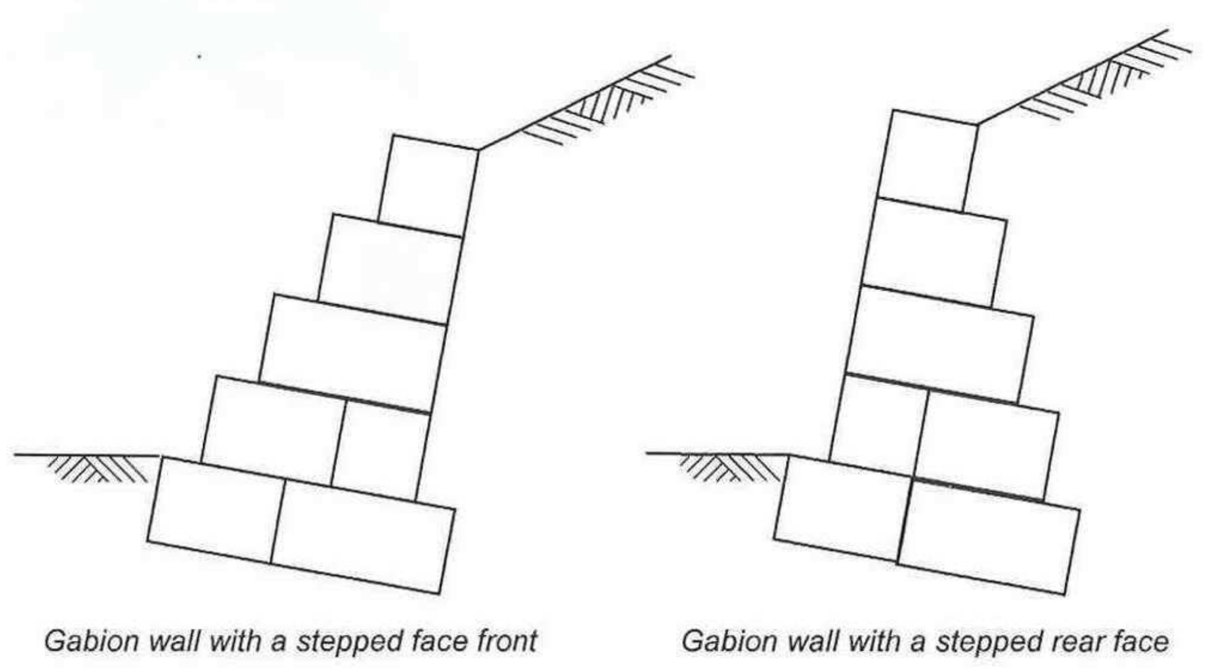
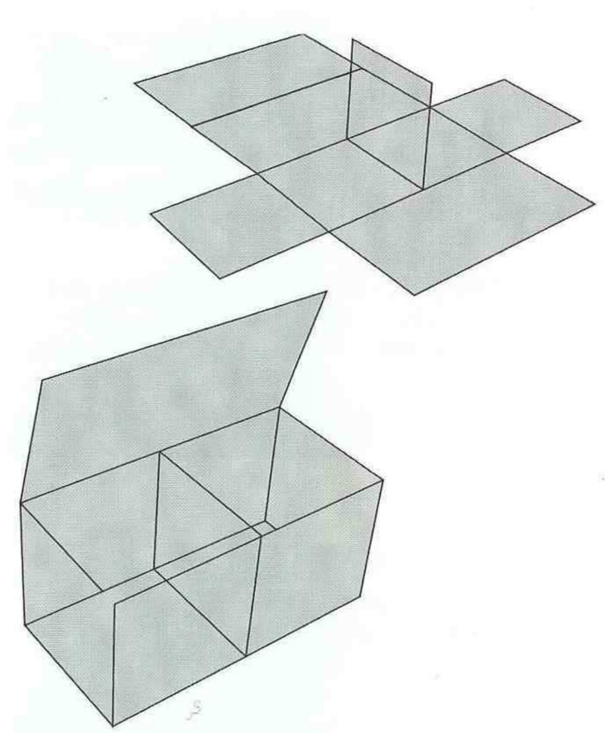


Figure 2: Tipical application of gabion walls (source www.gabbioni.it)

MITIGATION THROUGH REDUCTION OF HAZARD	
7 TRANSFER OF LOADS TO MORE COMPETENT STRATA	
7.2 GABION WALLS	
Figure 3: Tipical application of gabion walls (source www.protezionecivile.tn.it) 	Figure 4: Typical gabion as supplied (source: www.topfreebiz.com) 
Figure 5: Typical gabion assembly before filling with stone (source: BS 8002) 	Figure 6: Machine filling of gabions (source: www.degcostruzioni.com) 

MITIGATION THROUGH REDUCTION OF HAZARD			
7 TRANSFER OF LOADS TO MORE COMPETENT STRATA			
7.2 GABION WALLS			
APPLICABILITY			
Class	**Descriptor**	**Rating**	**Notes**
Type of movement (Cruden & Varnes, 1996)	Falls	0	Most suited to rotational or pseudo-rotational slides. May be useful to reduce toppling hazard in certain conditions
	Topples	2	
	Slides	8	
	Spreads	0	
	Flows	0	
Material	Earth	8	Mainly applicable to landslides involving earth and debris. Applicability in rock limited by typical slope geometry and failure mode
	Debris	6	
	Rock	4	
Depth of movement	Superficial (< 0.5 m)	4	Typically applicable to shallow to intermedite depth landslides.
	Shallow (0.5 to 3 m)	8	
	Medium (3 to 8 m)	8	
	Deep (8 to 15 m)	2	
	Very deep (> 15 m)	0	
Rate of movement (Varnes, 1978)	Moderately to fast	0	Should be carried out preferably on very or extremely slow landslides; with due care it can be carried out in slow landslides
	Slow	4	
	Very slow	8	
	Extremely slow	8	
Groundwater	Artesian	8	Applicable in all groundwater conditions. Stone filled gabion baskets are intrinsically free draining Adequate drainage must be provided at the interface between low permeability backfills, if any, and natural soil
	High	8	
	Low	8	
	Absent	8	
Surface water	Rain	6	Mechanical damege of facing from solid transport typically precludes use near torrents.
	Snowmelt	6	
	Localized	6	
	Stream	6	
	Torrent	0	
	River	6	
Maturity		8	Relatively simple technique. Potential benefits and limits of applicability are well established.
Reliability		8	The reliability of the technique depends on the reliability of the evaluation of the stability of the treated slope and of the foundations.
Implementation		8	Downgrade to 6 where pre-filled gabion baskets need to be lifted using cranes in confined workplaces or on steep slopes
Typical Cost		8	Low to moderate, provided local stone is used and the work does not involve diversion of major water courses or interference with existing infrastructure.

Note
Ratings are given on a scale of 1 to 10; the higher the grade, the most suitable is the specific method under consideration to use in landslides of the given characteristics, evaluated individually. Overall suitability to specific case under consideration may be obtained by a weighted average of these ratings, with user defined weights. Zero rating means "not applicable"

MITIGATION THROUGH REDUCTION OF HAZARD

7 TRANSFER OF LOADS TO MORE COMPETENT STRATA

7.2 GABION WALLS

References:

Chapman T., Taylor H., Nicholson D. (2000). “Modular Gravity Retaining Walls – Design Guidance”. Publication C516, CIRIA, London.

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MITIGATION THROUGH REDUCTION OF HAZARD

7 TRANSFER OF LOADS TO MORE COMPETENT STRATA

7.3 CRIB WALLS

Description

Crib walls comprise a grillage of header and stretcher elements placed on a firm foundation, usually of mass or reinforced concrete; the spaces between the grillage of header and stretcher elements are filled with free draining coarse grained materials (sand and gravel), which must be durable, e.g. resistant to erosion and frost; crib walls can be built with either the front face or the rear face stepped; it is desirable to incline the wall with an angle corresponding to 1 H: 4 V (Figure 1).

The header and stretcher elements can be made of reinforced concrete or timber and are designed to be interlocking; durability is provided by appropriate covering of the reinforcement in the concrete elements, or by treatment to timber elements. These elements are usually designed for manual handling; some more complex cellular systems exist where the header and stretcher elements are integrated such that they require a crane for lifting.

Wall can be made with one or multiple row of cribs at the base, depending on the height of the retaining structure to be formed. In order to facilitate construction, the backfill is placed and compacted keeping it to the same level reached by the wall. In placing the backfill within and behind the wall attention should be paid to avoid causing damage to the header and stretcher elements.

Crib walls are permeable and will allow retained fill to drain freely; where appropriate or necessary, surface and/or deep drainage systems will be provided to keep the backfill materials free from groundwater pressures.

Planting is possible, typically with livepole cuttings long enough to reach the backfill; the front face provides suitable anchorage for climbing or cascading vegetation. Advice on planting vegetation can be found in Coppin and Richards (1990). Care should be taken both in the choice of plants suitable for locations within, above or below the wall and for the suitability of the growing medium (usually loose topsoil or growbags) which may require special water retention measures.

Crib walls are susceptible to vandalism and accidental damage due to the small section size of the header and stretcher elements; timber and concrete elements are susceptible to fire, although it is unlikely that fire will be intense enough to cause more than superficial damage. Once the headers and stretchers have been erected, it is possible to fill a crib wall with lean mix concrete, making it more akin to a masonry wall; in this case, the free-draining nature of the wall is lost and a drainage system may have to be incorporated to prevent the build-up of groundwater pressures behind the wall.

Design

The wall specification should stipulate the materials to be considered for filling within and behind the wall.

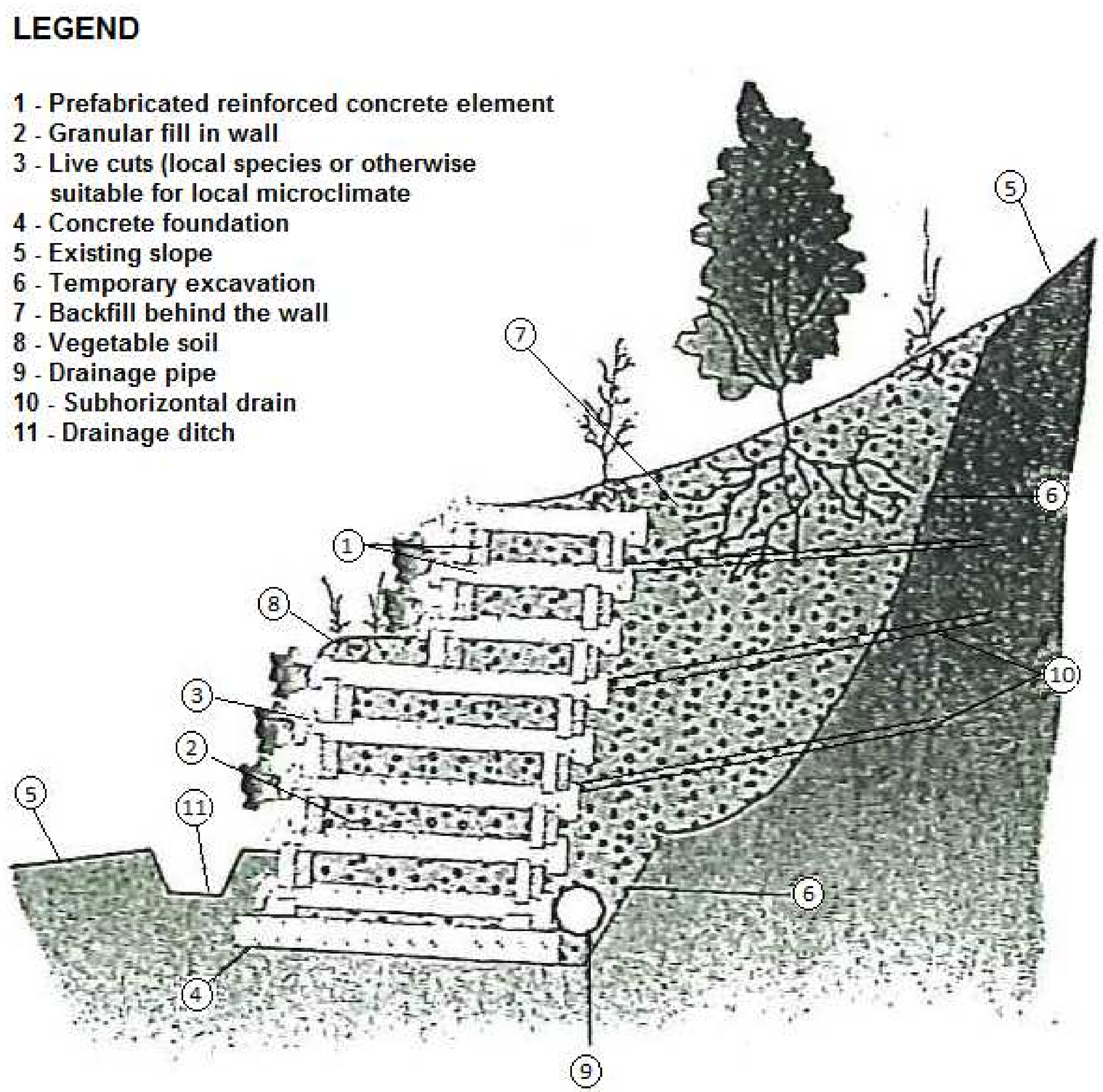
The properties of the backfill will depend on whether or not locally-won backfill is to be used, and if the material is required to be free-draining. Optimum backfill is: easy to compact, giving high strength and stiffness; and free-draining, to minimize the build-up of groundwater pressure. Backfill should not include: natural or contaminated soil which will be chemically aggressive; frozen materials; degradable materials such as topsoil, peat, wood, vegetation, etc.; materials which could be toxic, dangerous or prone to spontaneous combustion; soluble material or collapsible soils. The use of clays prone to swelling should be carefully considered as they can exert very high pressures on the back of retaining walls; the same applies for materials derived from argillaceous rocks such as shales and mudstones. Care should be taken to ensure that the infill material cannot escape from the crib wall. Sometimes it needs to be retained using geotextile.

Walls design shall put special consideration on aspects related to water pressure and drainage. Rationale and details for drainage systems can be found for example in Geotechnical Engineering Office (1993) and Chapman et al. (2000).

The following ultimate limit states (ULS) need to be verified:

- Bearing resistance failure at the base of the wall;
- Sliding failure at the base of the wall;
- Failure by toppling of the wall;
- Loss of overall stability around the wall;
- Overall stability of the slope, including the wall;
- Unacceptable leakage through or beneath the wall;
- Unacceptable transport of soil grains through or beneath the wall;
- Internal stability. The main aspect of internal stability that will concern designers is checking that sliding and overturning failures cannot occur at various levels within the wall. BS 8002 Clause 4.2.7.2.3 warns against the use of crib walls to retain unstable slopes; this is because the crib walls will not offer much resistance to failure planes passing through it. The detailed design of reinforced concrete elements will normally be undertaken by specialist suppliers. Information on the forces for which the crib modules should be designed is given in BS 8002 Cl. 4.2.7.4.2 and in greater detail in BD 68/97.

Figure 1: Typical details of gabion walls (source Chapman et al., 2000)



MITIGATION THROUGH REDUCTION OF HAZARD

7 TRANSFER OF LOADS TO MORE COMPETENT STRATA

7.3 CRIB WALLS

Figure 2: Timber crib wall with quick-set live cuttings (source: SGI-MI)

Figure 3: Connection detail in timber crib wall (source: SGI-MI)

Figure 4: Concrete crib wall (source: E Zimbres, Rio de Janeiro State University, through Wikimedia Commons)

Figure 5: Concrete crib wall with multiple rows (source: www.gibbonscontractors.co.nz)

MITIGATION THROUGH REDUCTION OF HAZARD			
7 TRANSFER OF LOADS TO MORE COMPETENT STRATA			
7.3 CRIB WALLS			
APPLICABILITY			
Class	**Descriptor**	**Rating**	**Notes**
Type of movement (Cruden & Varnes, 1996)	Falls	0	Most suited to rotational or pseudo-rotational slides. May be useful to reduce toppling hazard in certain conditions
	Topples	2	
	Slides	8	
	Spreads	0	
	Flows	0	
Material	Earth	8	Mainly applicable to landslides involving earth and debris. Applicability in rock limited by typical slope geometry and failure mode
	Debris	6	
	Rock	4	
Depth of movement	Superficial (< 0.5 m)	0	Typically applicable to shallow to intermedite depth landslides. Minimum size of elements makes this approach impractical for superficial landslides
	Shallow (0.5 to 3 m)	8	
	Medium (3 to 8 m)	8	
	Deep (8 to 15 m)	2	
	Very deep (> 15 m)	0	
Rate of movement (Varnes, 1978)	Moderately to fast	0	Should be carried out preferably on very or extremely slow landslides; with due care it can be carried out in slow landslides
	Slow	4	
	Very slow	8	
	Extremely slow	8	
Groundwater	Artesian	8	Applicable in all groundwater conditions. Stone filled crib walls are intrinsically free draining Adequate drainage must be provided at the interface between low permeability backfills, if any, and natural soil
	High	8	
	Low	8	
	Absent	8	
Surface water	Rain	6	Not applicable in contact with watercourses.
	Snowmelt	6	
	Localized	6	
	Stream	0	
	Torrent	0	
	River	0	
Maturity		8	Relatively simple technique. Potential benefits and limits of applicability are well established.
Reliability		8	The reliability of the technique depends on the reliability of the evaluation of the stability of the treated slope and of the foundations.
Implementation		8	Downgrade to 6 where elements need to be lifted using cranes in confined workplaces or on steep slopes
Typical Cost		6	Moderate, provided local stone is used and the work does not involve diversion of major water courses or interference with existing infrastructure.

Note
Ratings are given on a scale of 1 to 10; the higher the grade, the most suitable is the specific method under consideration to use in landslides of the given characteristics, evaluated individually. Overall suitability to specific case under consideration may be obtained by a weighted average of these ratings, with user defined weights. Zero rating means "not applicable"

MITIGATION THROUGH REDUCTION OF HAZARD

7 TRANSFER OF LOADS TO MORE COMPETENT STRATA

7.3 CRIB WALLS

References:

BD 68/97 ”Crib retaining walls”

BS 8002 (1994) ”Code of Practice for Earth Retaining Structures”.

Chapman T., Taylor H., Nicholson D. (2000). “Modular Gravity Retaining Walls – Design Guidance”. Publication C516, CIRIA, London.

Coppin N.J., Richards I.G. (1990) ”Use of vegetation in civil engineering” CIRIA Book 10, CIRIA/Butterworths, London .

Geotechnical Engineering Office (1993) ”Geoguide 1 – Guide to Retaining Wall Design” Civil Engineering Department, The Government of the Hong Kong, Special Administrative Region.

MITIGATION THROUGH REDUCTION OF HAZARD

7 TRANSFER OF LOADS TO MORE COMPETENT STRATA

7.4 DRYSTACK MASONRY WALLS

Description

Drystack masonry walls consist of precast concrete special blocks and occasionally bricks designed to interlock with each other and to produce a solid wall face; interlock provides adequate shear resistance between each layer of blocks and assists accurate placing of successive layers of blocks.

Drystack masonry walls must be constructed on a mass or reinforced concrete foundation. In order to provide additional stability it is possible to build the walls thicker at the base. It is also prudent to specify the top layers of blocks to have some form of mortar pointing, adhesive or capping to avoid them being dislodged by vandals.

The individual blocks are usually designed to be placed by manual handling. The walls can be constructed vertically or with a batter to provide better stability for greater heights. In order to facilitate construction the backfill is placed and compacted keeping it to the same level reached by the wall.

Most of the drystack masonry walls are relatively free-draining; however, where appropriate or necessary, surface and/or deep drainage systems will be provided to keep the backfill materials free from groundwater pressures.

Drystack masonry walls may be used in association with top, bottom or wall –face planting; wall-face planting may be in spaces provided by open joints filled with a suitable growing medium. Advice on planting vegetation can be found in Coppin and Richards (1990). Care should be taken both in the choice of plants suitable for locations within, above or below the wall and for the suitability of the growing medium (usually loose topsoil or growbags) which may require special water retention measures.

Design

Drystack masonry walls should be designed as gravity mass walls.

The wall specification should stipulate the materials to be considered for filling behind the wall

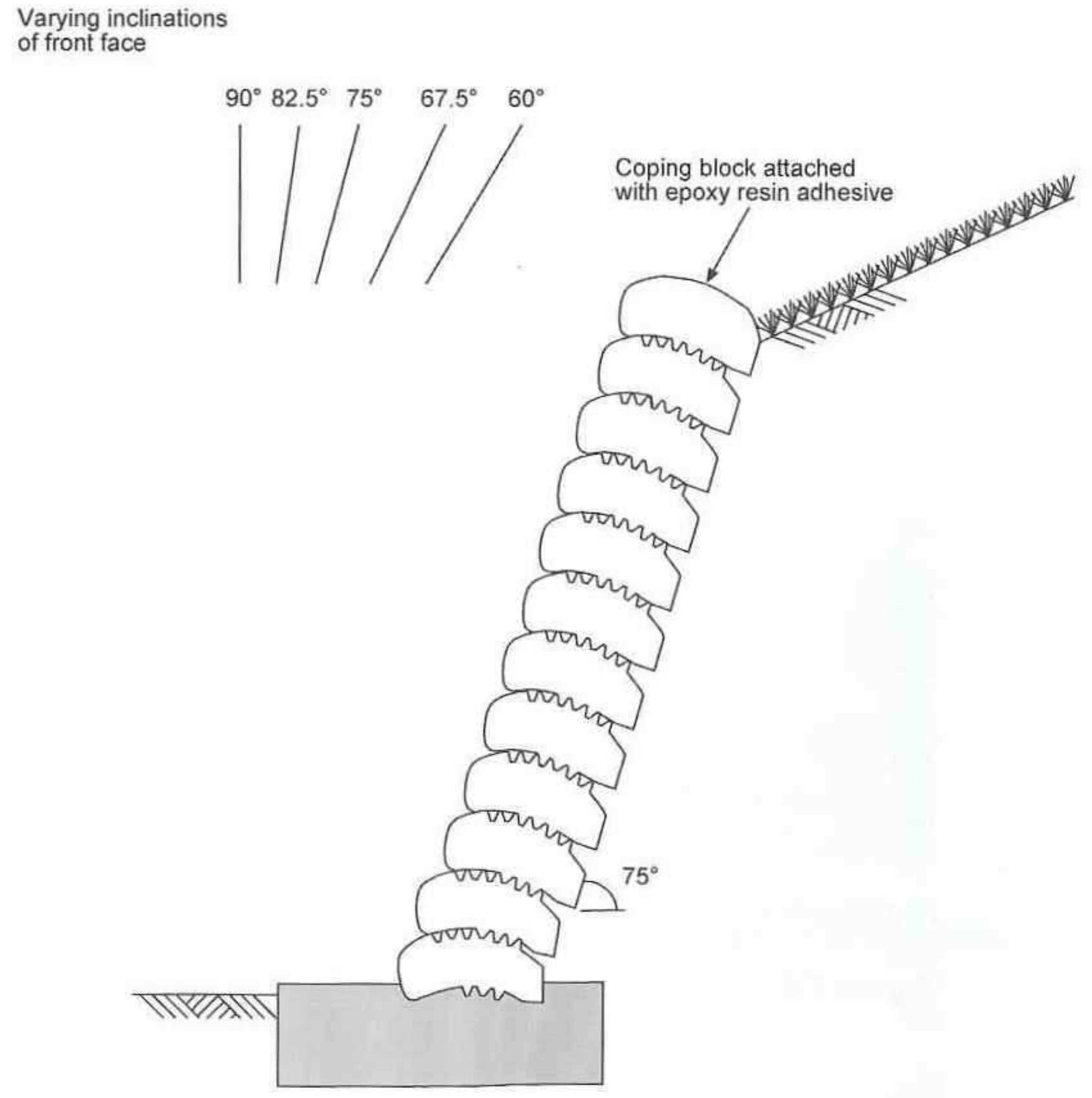
The properties of the backfill will depend on whether or not locally-won backfill is to be used, and if the material is required to be free-draining. Optimum backfill is: easy to compact, giving high strength and stiffness; and free-draining, to minimize the build-up of groundwater pressure. Backfill should not include: natural or contaminated soil which will be chemically aggressive; frozen materials; degradable materials such as topsoil, peat, wood, vegetation, etc.; materials which could be toxic, dangerous or prone to spontaneous combustion; soluble material or collapsible soils. The use of clays prone to swelling should be carefully considered as they can exert very high pressures on the back of retaining walls; the same applies for materials derived from argillaceous rocks such as shales and mudstones.

Walls design shall put special consideration on aspects related to water pressure and drainage. Rationale and details for drainage systems can be found for example in Geotechnical Engineering Office (1993) and Chapman et al. (2000).

The following ultimate limit states (ULS) need to be verified:

- Bearing resistance failure at the base of the wall;
- Sliding failure at the base of the wall;
- Failure by toppling of the wall;
- Loss of overall stability around the wall;
- Overall stability of the slope, including the wall;
- Unacceptable leakage beneath the wall;
- Unacceptable transport of soil grains beneath the wall;
- Internal stability. The resultant force at any horizontal sections shall be within the middle third of the section. Checks should be made of horizontal sections above the base of the wall that there is adequate resistance to sliding.

Figure 1: Typical example of drystack masonry wall (source Chapman et al., 2000)



MITIGATION THROUGH REDUCTION OF HAZARD			
7 TRANSFER OF LOADS TO MORE COMPETENT STRATA			
7.4 DRYSTACK MASONRY WALLS			
APPLICABILITY			
Class	**Descriptor**	**Rating**	**Notes**
Type of movement (Cruden & Varnes, 1996)	Falls	0	Only suited to rotational or pseudo-rotational slides which are fully stabilized with no further movement
	Topples	0	
	Slides	4	
	Spreads	0	
	Flows	0	
Material	Earth	8	Mainly applicable to landslides involving earth and debris. Applicability in rock limited by typical slope geometry and failure mode
	Debris	6	
	Rock	4	
Depth of movement	Superficial (< 0.5 m)	8	Typically applicable to shallow landslides, fully stabilized.
	Shallow (0.5 to 3 m)	8	
	Medium (3 to 8 m)	2	
	Deep (8 to 15 m)	0	
	Very deep (> 15 m)	0	
Rate of movement (Varnes, 1978)	Moderately to fast	0	Should be carried out preferably on very or extremely slow landslides which become fully stabilized.
	Slow	0	
	Very slow	6	
	Extremely slow	8	
Groundwater	Artesian	8	Applicable in all groundwater conditions. Adequate drainage must be provided to wall and at the interface between low permeability backfills, if any, and natural soil
	High	8	
	Low	8	
	Absent	8	
Surface water	Rain	6	Not applicable in contact with watercourses.
	Snowmelt	6	
	Localized	6	
	Stream	0	
	Torrent	0	
	River	0	
Maturity		6	Relatively simple technique, but applicability in landslide remediation must be proven.
Reliability		4	Reliability penalized by susceptibility to loss of integrity on further movement.
Implementation		8	Downgrade to 6 where elements need to be lifted using cranes in confined workplaces or on steep slopes
Typical Cost		8	Low to moderate, provided the work does not involve diversion of major water courses or interference with existing infrastructure.

Note
Ratings are given on a scale of 1 to 10; the higher the grade, the most suitable is the specific method under consideration to use in landslides of the given characteristics, evaluated individually. Overall suitability to specific case under consideration may be obtained by a weighted average of these ratings, with user defined weights. Zero rating means "not applicable"

MITIGATION THROUGH REDUCTION OF HAZARD

7 TRANSFER OF LOADS TO MORE COMPETENT STRATA

7.4 DRYSTACK MASONRY WALLS

References:

Chapman T., Taylor H., Nicholson D. (2000). “Modular Gravity Retaining Walls – Design Guidance”. Publication C516, CIRIA, London.

Coppin N.J., Richards I.G. (1990) ”Use of vegetation in civil engineering” CIRIA Book 10, CIRIA/Butterworths, London .

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MITIGATION THROUGH REDUCTION OF HAZARD

7 TRANSFER OF LOADS TO MORE COMPETENT STRATA

7.5 MASS CONCRETE OR MASONRY WALLS

Description

Mass concrete, stone and masonry walls are possibly the oldest form of retaining structures. They are found in archaeological sites from all ages round the world. They punctuate the landscape to the extent that they often go unnoticed (Figures 1 and 2).

Masonry walls are made with bricks, blocks, natural or manufactured stones conventionally bedded with mortar, but are otherwise similar to mass concrete walls; all these walls are built on a mass or reinforced concrete foundations; they can be provided with fins or reveals to improve their overturning resistances and may be built conveniently to curves and irregular plan forms.

BS 8002 Clause 4.2.4 suggests that the simple stem walls are suitable for retaining heights up to 1.5 m while greater heights can be accommodated by stepped or buttressed walls.

In order to avoid water saturation and possible frost damage, masonry walls shall be provided with protective measures (coping, drainage, damp-proof courses and water-proofing, see for example BS 5628-Part 3, 1985) and frost resistant mortar, bricks, blocks or stones.

Masonry walls are constructed in panels, typically 10 to 15 m in length between joints; joints should be detailed in such a way to prevent unattractive vertical lines of seepage in the wall face. Panels of this length may require horizontal reinforcement. Backfill can be placed only when mortar has had time to gain appropriate strength. Durability is addressed in BS 5628 and Thomas (1996); particular attention should be paid to sulphate attack.

Masonry walls can be provided with decoration. Examples are shown in the "Brick Development Association publication by Haseltine and Tutt (1991).

Design

The wall specification should stipulate the materials to be considered for filling behind the wall.

The properties of the backfill will depend on whether or not locally-won backfill is to be used, and if the material is required to be free-draining. Optimum backfill is: easy to compact, giving high strength and stiffness; and free-draining, to minimize the build-up of groundwater pressure. Backfill should not include: natural or contaminated soil which will be chemically aggressive; frozen materials; degradable materials such as topsoil, peat, wood, vegetation, etc.; materials which could be toxic, dangerous or prone to spontaneous combustion; soluble material or collapsible soils. The use of clays prone to swelling should be carefully considered as they can exert very high pressures on the back of retaining walls; the same applies for materials derived from argillaceous rocks such as shales and mudstones.

These walls should be designed as gravity mass walls (see for example BS 8002; BS 5628; Geotechnical Engineering Office, 1993 and Chapman et al., 2000). By far the most common form of masonry wall is the simple brick wall; the design of unreinforced brickwork retaining walls is addressed by Haseltine and Tutt (1991).

Thomas (1996) gives guidance on the design and specification of all types of masonry walls, giving details from relevant standards.

Walls design shall put special consideration on aspects related to water pressure and drainage. Rationale and details for drainage systems can be found for example in Geotechnical Engineering Office (1993) and Chapman et al. (2000).

Special attention is also required in the design, construction and maintenance of masonry walls in seismic areas, since earthquake shaking can induce internal failure besides the global mechanisms that are normally considered in design (Figure 3 and 4).

The following ultimate limit states (ULS) need to be verified:

- Bearing resistance failure at the base of the wall;
- Sliding failure at the base of the wall;
- Failure by toppling of the wall;
- Loss of overall stability around the wall;
- Overall stability of the slope, including the wall;
- Unacceptable leakage beneath the wall;
- Unacceptable transport of soil grains beneath the wall;
- Internal stability. This aspect is covered by BS 5628 parts 1 and 2, and simple design rules are given by Haseltine and Tutt (1991); walls should be designed to resist overturning; buttresses and fins can be used to provide resistance in circumstances where their use would not conflict with other requirements.

Figure 1:Stone masonry retaining walls at Machu Pichu (source. www.travel.webshot.com)

Figure 2: Stone Retaining walls in Brazil (source: E Zimbres, Rio de Janeiro State University, in Wikimedia Commons)

Figure 3: retaining wall in L'Aquila: no or tolerable damage in 2009 earthquake (source www.geerassociation.org)

Figure 4: Retaining wall near L'Aquila. Local damaged in 2009 earthquake (source www.geerassociation.org)

MITIGATION THROUGH REDUCTION OF HAZARD			
7 TRANSFER OF LOADS TO MORE COMPETENT STRATA			
7.5 MASS CONCRETE OR MASONRY WALLS			
APPLICABILITY			
Class	**Descriptor**	**Rating**	**Notes**
Type of movement (Cruden & Varnes, 1996)	Falls	0	Only suited to rotational or pseudo-rotational slides which are fully stabilized with no further movement
	Topples	0	
	Slides	6	
	Spreads	0	
	Flows	0	
Material	Earth	8	Mainly applicable to landslides involving earth and debris. Applicability in rock limited by typical slope geometry and failure mode
	Debris	6	
	Rock	4	
Depth of movement	Superficial (< 0.5 m)	4	Typically applicable to shallow landslides, fully stabilized.
	Shallow (0.5 to 3 m)	8	
	Medium (3 to 8 m)	6	
	Deep (8 to 15 m)	0	
	Very deep (> 15 m)	0	
Rate of movement (Varnes, 1978)	Moderately to fast	0	Should be carried out preferably on very or extremely slow landslides which become fully stabilized.
	Slow	0	
	Very slow	6	
	Extremely slow	8	
Groundwater	Artesian	8	Applicable in all groundwater conditions. Adequate drainage must be provided to wall and at the interface between low permeability backfills, if any, and natural soil
	High	8	
	Low	8	
	Absent	8	
Surface water	Rain	6	Applicable in contact with watercourses, but construction requires temporary diversion/exclusion and foundations must be protected against scour.
	Snowmelt	6	
	Localized	6	
	Stream	6	
	Torrent	4	
	River	6	
Maturity		8	Relatively simple technique, Potential benefits and limits of applicability are well established.
Reliability		6	Reliability penalized by susceptibility to loss of integrity on further movement.
Implementation		8	Downgrade to 6 where work involves heavy lifting using cranes in confined workplaces or on steep slopes
Typical Cost		8	Low to moderate, provided the work does not involve diversion of major water courses or interference with existing infrastructure.

Note
Ratings are given on a scale of 1 to 10; the higher the grade, the most suitable is the specific method under consideration to use in landslides of the given characteristics, evaluated individually. Overall suitability to specific case under consideration may be obtained by a weighted average of these ratings, with user defined weights. Zero rating means "not applicable"

MITIGATION THROUGH REDUCTION OF HAZARD

7 TRANSFER OF LOADS TO MORE COMPETENT STRATA

7.5 MASS CONCRETE OR MASONRY WALLS

References:

BS 8002 (1994) ”Code of Practice for Earth Retaining Structures”.

BS 5628, Part 1 (1992) ”Code of practice for use of masonry - Structural use of unreinforced masonry”.

BS 5628, Part 2 (1995) ”Code of practice for use of masonry - Structural use of reinforced and prestressed masonry”.

BS 5628, Part 3 (1985) ”Code of practice for use of masonry - Materials and components, design and workmanship”.

Chapman T., Taylor H., Nicholson D. (2000). “Modular Gravity Retaining Walls – Design Guidance”. Publication C516, CIRIA, London.

Geotechnical Engineering Office (1993) ”Geoguide 1 – Guide to Retaining Wall Design” Civil Engineering Department, The Government of the Hong Kong, Special Administrative Region.

Haseltine B.A., Tutt J.N. (1991) ” The design of brickwark retaining walls” The Brick Development Association Design Guide n° 2, revised edition.

Thomas K. (1996) ”Masonry walls, specification and design” Butterworth Heinemann.

MITIGATION THROUGH REDUCTION OF HAZARD

7 TRANSFER OF LOADS TO MORE COMPETENT STRATA

7.6 REINFORCED CONCRETE STEM WALLS

Description

Reinforced concrete stem walls, called also cantilever walls or gravity cantilever walls, are L-shaped or inverted T-shaped walls which rest on the ground and act, from a geotechnical stability point of view, in conjunction with the mass of the retained fill above the foundation element. Tall or heavy duty walls may incorporate additional buttresses or slabs.

These walls may be cast in-situ or prefabricated; cast in-situ walls requires a range of site craft skills, but can be more effective in difficult situations, where the foundation soil is poor or where interaction with other structures is required; prefabricated walls, which are manufactured in a wide range of heights, can usually be installed without a specialist labour force and the construction process is relatively fast, but they require good access and the use of a crane for offloading and installation.

Different finishes can be applied, especially to precast walls, using stone or matrix formwork.

Reinforced concrete elements should have vertical joints. BS 8002 Clause 4.3.1.4.6 recommends that where necessary (generally where water penetration from the retained fill through the wall joints would be unsightly or could damage a facing structure), "*joints should be lined with a resilient jointing materials about 10 to 20 mm thick and sealed with a proprietary sealing compound. Dependent upon the groundwater present, waterbars may be also required*".

A drainage layer is normally installed on the back of the wall to limit pressures on the stem. Additionally, where appropriate/necessary to keep the backfill materials free from groundwater pressures, surface and/or deep drainage systems will be foreseen.

Standard precautions for ensuring the durability of reinforced concrete should be followed (see for example BS 8110, EC2, BS 5400- Part 4).

Design

The wall specification should stipulate the materials to be considered for filling behind the wall.

The properties of the backfill will depend on whether or not locally-won backfill is to be used, and if the material is required to be free-draining. Optimum backfill is: easy to compact, giving high strength and stiffness; and free-draining, to minimize the build-up of groundwater pressure. Backfill should not include: natural or contaminated soil which will be chemically aggressive; frozen materials; degradable materials such as topsoil, peat, wood, vegetation, etc.; materials which could be toxic, dangerous or prone to spontaneous combustion; soluble material or collapsible soils. The use of clays prone to swelling should be carefully considered as they can exert very high pressures on the back of retaining walls; the same applies for materials derived from argillaceous rocks such as shales and mudstones.

From a geotechnical point of view, these walls should be designed as a gravity mass walls considering the backfill on the foundation slab as an integral part of the wall (see for example BS 8002; Geotechnical Engineering Office, 1993 and Chapman et al., 2000); the vertical plane on which the earth pressure are evaluated is that through the back of the heel, not the stem. Even where "active" earth pressure conditions are applicable on the upslope face of the wall for geotechnical design, structural design should be based on "at rest" conditions, since normally the stem and the connection between the stem and the base will be too stiff to allow sufficient relative movement between wall and backfill to generate "active" conditions. Higher earth pressures may occur as a result of compaction, especially on the upper portion of the stem (Ingold, 1979; Duncan et al., 1991).

Wall design shall pay special attention to aspects related to water pressure and drainage. Rationale for drainage systems and related details can be found for example in Geotechnical Engineering Office (1993) and Chapman et al. (2000).

The following ultimate limit states (ULS) need to be verified:

- Bearing resistance failure at the base of the wall;
- Sliding failure at the base of the wall;
- Failure by toppling of the wall;
- Loss of overall stability around the wall;
- Overall stability of the slope, including the wall;
- Unacceptable leakage beneath the wall;
- Unacceptable transport of soil grains beneath the wall;
- Internal stability. The forces to be used for the design of the stem and heel will be evaluated according to guidance provided by for example BS 8002, EC2, BS 8110 (see also Geotechnical Engineering Office, 1993 and Chapman et al., 2000).

Figure 1: Typical in-situ r.c. stem wall
(Source www.mailingmaggioli.it))

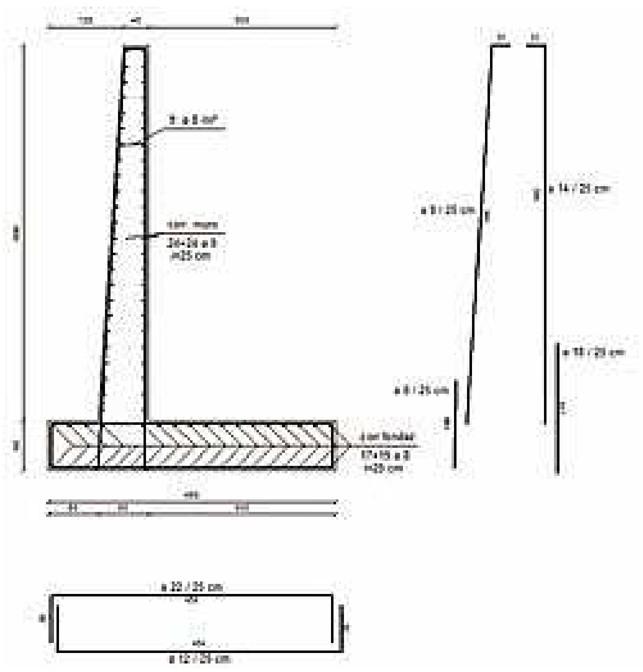


Figure 2: In-situ r.c. stem wall construction
(Source www.villacostruzioni.com)

Figure 3: Typical prefabricated buttressed stem wall
(Source: www.tensiter.com)

Figure 4: mixed in-situ and prefabricated stem wall
(Source: www.villacostruzioni.com)

MITIGATION THROUGH REDUCTION OF HAZARD			
7 TRANSFER OF LOADS TO MORE COMPETENT STRATA			
7.6 REINFORCED CONCRETE STEM WALLS			
APPLICABILITY			
Class	**Descriptor**	**Rating**	**Notes**
Type of movement (Cruden & Varnes, 1996)	Falls	0	Only suited to rotational or pseudo-rotational slides which are fully stabilized with no further movement
	Topples	0	
	Slides	6	
	Spreads	0	
	Flows	0	
Material	Earth	8	Mainly applicable to landslides involving earth and debris. Applicability in rock limited by typical slope geometry and failure mode
	Debris	6	
	Rock	4	
Depth of movement	Superficial (< 0.5 m)	0	Typically applicable to shallow to landslides, fully stabilized.
	Shallow (0.5 to 3 m)	8	
	Medium (3 to 8 m)	6	
	Deep (8 to 15 m)	0	
	Very deep (> 15 m)	0	
Rate of movement (Varnes, 1978)	Moderately to fast	0	Should be carried out preferably on very or extremely slow landslides which become fully stabilized.
	Slow	0	
	Very slow	6	
	Extremely slow	8	
Groundwater	Artesian	8	Applicable in all groundwater conditions. Adequate drainage must be provided to wall and at the interface between low permeability backfills, if any, and natural soil
	High	8	
	Low	8	
	Absent	8	
Surface water	Rain	6	Applicable in contact with watercourses, but construction requires temporary diversion/exclusion and foundations must be protected against scour.
	Snowmelt	6	
	Localized	6	
	Stream	6	
	Torrent	4	
	River	6	
Maturity		8	Relatively simple technique, Potential benefits and limits of applicability are well established.
Reliability		6	Reliability penalized by susceptibility to loss of integrity on further movement.
Implementation		8	Downgrade to 6 where work involves heavy lifting using cranes in confined workplaces or on steep slopes
Typical Cost		6	Moderate, provided the work does not involve diversion of major water courses or interference with existing infrastructure.

Note
Ratings are given on a scale of 1 to 10; the higher the grade, the most suitable is the specific method under consideration to use in landslides of the given characteristics, evaluated individually. Overall suitability to specific case under consideration may be obtained by a weighted average of these ratings, with user defined weights. Zero rating means "not applicable"

MITIGATION THROUGH REDUCTION OF HAZARD

7 TRANSFER OF LOADS TO MORE COMPETENT STRATA

7.6 REINFORCED CONCRETE STEM WALLS

References:

BS 5400 – Part 4 (1990) ”Steel, concrete and composite bridges, code of practice for design of concrete bridges”.

BS 8002 (1994) ”Code of Practice for Earth Retaining Structures”.

BS 8110 – Part 1 (1997) ”Structural use of concrete, code of practice for design and construction”.

BS 8110 – Part 2 (1965) ”Structural use of concrete, code of practice for special circumstances”.

Chapman T., Taylor H., Nicholson D. (2000). “Modular Gravity Retaining Walls – Design Guidance”. Publication C516, CIRIA, London.

Duncan J.M., Williams G.W., Sehn A. L., Seed R.B. (1991). ”Estimation Earth Pressures Due to Compaction”. Journal of Geotechnical Engineering, Vol. 117, No. 12, 1833-1847; discussion and closure, Vol. 119, No. 7, July 1993, 1162-1177

EC2 ”Design of concrete structures”.

Geotechnical Engineering Office (1993) ”Geoguide 1 – Guide to Retaining Wall Design” Civil Engineering Department, The Government of the Hong Kong, Special Administrative Region.

Ingold, T.S., (1979) The effects of compaction on retaining walls, Gèotechnique, 29, p265-283

ANNEX B

MITIGATION THROUGH REDUCTION OF EXPOSED POPULATIONS

FACT SHEETS

MITIGATION THROUGH REDUCTION OF EXPOSED POPULATIONS

FACT SHEET 1

EARLY WARNING SYSTEMS

MITIGATION THROUGH REDUCTION OF EXPOSED POPULATIONS

1 EARLY WARNING SYSTEMS

1.1 DISPLACEMENTS MONITORING

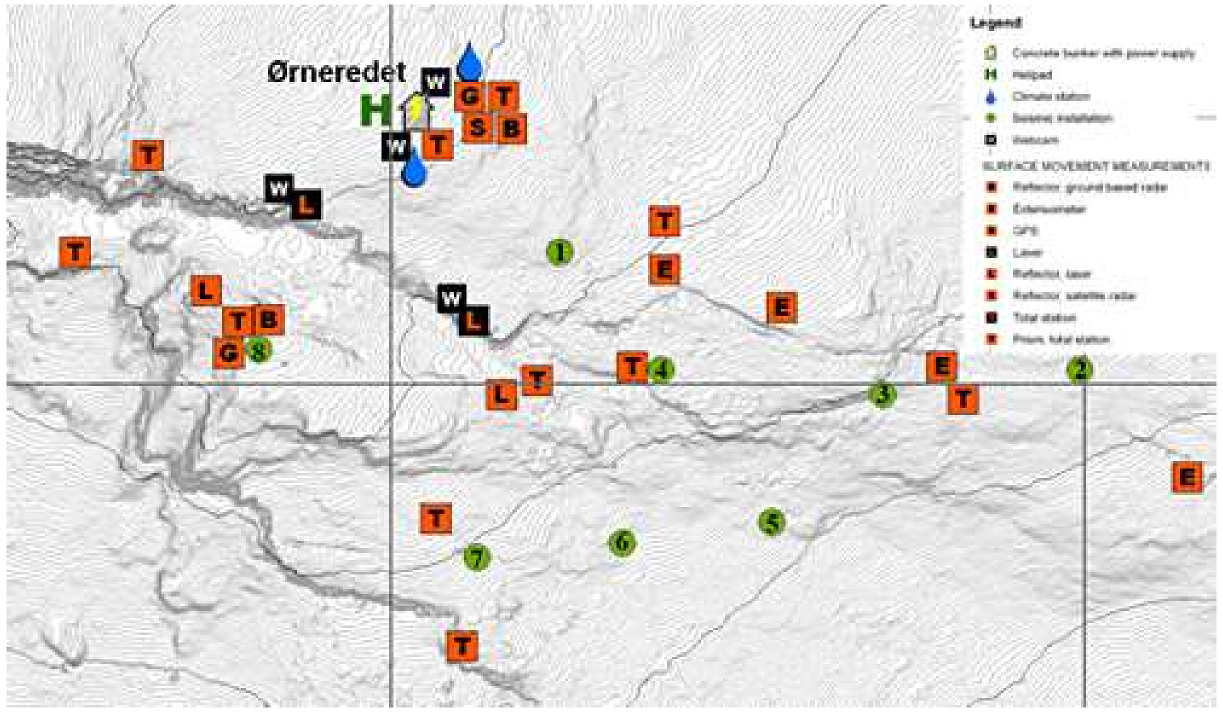
Description
In some situations, landslides are too large and/or high to think to remove completely the hazard with standard techniques previously described in the Annex A, such as the modification of the slope geometry or the installation of anchors. It is particularly highlighted by major rockslides as the famous ones of Frank slide (Canada), Åknes or Nordnes rockslides (Norway), Ancona and Beauregard landslides (Italy) where risk management strategies are presently mainly based on early warning systems. Nevertheless, as large rockslides can seriously threaten people and infrastructures, new kinds of measures have to be achieved in order to reduce the risk, especially in term of human lives. One way to do that is to reduce the exposition thanks to appropriate evacuation plans. Indeed, many researches aim to build reliable early warning systems in order to be able to alert and evacuate endangered populations as soon as monitored displacements reach previously established velocity thresholds.

Design
There is no rule in the design of early warning systems; it can change a lot from a site to another one, depending of the type of landslide, the failure mechanismes, the size of the instability, the available technologies and resources, etc… Nevertheless, two fundamental rules have to be considered:
- It is crucial to have a complete view and understanding of the intability (i.e. extends of unstable areas, failure mechanismes, etc…) in order to setup systems at a relevant location in the field and to fix appropriate threshold parameters, such as velocity or level of water table.
- To achieve reliable and robust systems, the implementation of complementary devices is a real key point. Usually, simple and robust ones are prefered, such as extensometers, tiltmeters or automatic distancemeters. Nevertheless, they can be linked with more complexe instruments providing additionnal information about displacements in three dimensions, usually with GNSS antennas, and about deformations of the entire instability, such as ground-based radar interferometry (GB-InSAR) devices which presently gain in importance and is steadily used.

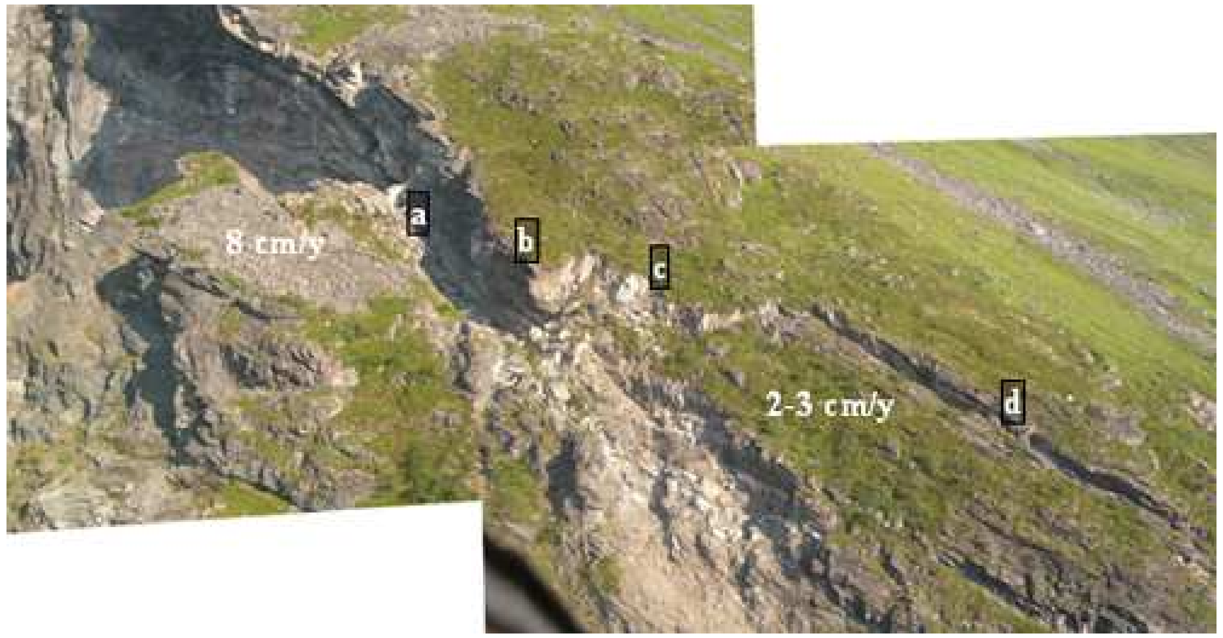
SafeLand deliverables dedicated to early warning systems
Considering the complexity and the importance of the topic, three distinct deliverables within the SafeLand European project are linked and dedicated to the design of early warning systems. As developing more this topic in this compedium would be too long, we strongly recommend to consult them:

1) SafeLand deliverable 4.1, 2010. Review of Techniques for Landslide Detection, Fast Characterization, Rapid Mapping and Long-Term Monitoring. Edited for the SafeLand European project by Michoud C., Abellán A., Derron M.-H. and Jaboyedoff M. Available at http://www.safeland-fp7.eu
2) SafeLand deliverable 4.4, 2011. Guidelines for the selection of appropriate remote sensing technologies for monitoring different types of landslides. Edited for the SafeLand European project by Stumpf A. and Kerle N. Available at http://www.safeland-fp7.eu
3) SafeLand deliverable 4.8, 2012. Guidelines for the monitoring and early warning systems in Europe – Design and required technologies. Edited for the SafeLand European project by the ICG. Available at http://www.safeland-fp7.eu

Figure 1: Current monitoring devices setup for the Åknes rockslide (available on the NORSAR website: www.norsar.no/c-48-Aaknes-Rock-Slide.aspx).



Picture 1: Back scarp of the Åknes rockslide with the position of differents monitoring instruments: (a) geophone, (b) laser and webcam and (d) extensometer (available on the NORSAR website: www.norsar.no/c-48-Aaknes-Rock-Slide.aspx)



MITIGATION THROUGH REDUCTION OF EXPOSED POPULATIONS

1 EARLY WARNING SYSTEMS

1.1 DISPLACEMENTS MONITORING

References:

Agliardi F., Crosta G.B. and Frattini P. (2010). Forecasting the Failure of Large Landslides for Early Warning: Issues and Directions. In: Landslide Monitoring Technologies and Early Warning Systems, Proceeding of the Workshop within the frame of the FP7 “SafeLand” project, February 24th, 2010, Vienna, edited by Supper R. and Baron I., 48-49.

Barla G., Antolini F., Barla M., Mensi E. And Piovani G. (in press). Monitoring of the Beauregard landslide (Aosta Valley, Italy) using advanced and conventional techniques. Engineering Geology, 18 p.

Bjorgan G. and Froese C.R. (2008). Combined GPS and EDM monitoring on Turtle Mountain, Alberta. GeoEdmonton2008, 1236-1242.

Blikra L.H. (2008). The Åknes rockslide; monitoring, threshold values and early-warning, in: Landslides and Engineered Slopes. From the Past to the Future Proceedings of the 10th International Symposium on Landslides and Engineered Slopes, 30 June–4 July 2008, Xi’an, China, edited by: Chen, Z., Zhang, J., Li, Z., Wu, F., and Ho, K., Taylor & Francis Group, London, 2, 1089−1094.

Cardellini S. and Osimani P. (2010). Living with Landslides: the Ancona Case History and Early Warning System. In: Landslide Monitoring Technologies and Early Warning Systems, Proceeding of the Workshop within the frame of the FP7 “SafeLand” project, February 24th, 2010, Vienna, edited by Supper R. and Baron I., 61-66.

Crosta G.B. and Agliardi F. (2002). How to obtain alert velocity threshold for large rockslides, Pysics and Chemistry of the Earth, 27, 1557-1565.

Froese C.R., Carter G., Langenberg W. and Moreno F. (2006). Emergency Response Planning for a Second Catastrophic Rock Slide at Turtle Mountain, Alberta. 1st Specialty Conference on Disaster Mitigation, 23-26 May 2006, Calgary, Alberta, Canada, 10 p.

Froese C.R., Moreno F., Jaboyedoff M. and Cruden D.M. (2009). 25 years of movement monitoring on South Peak, Turtle Mountain: understanding the hazard. Can. Geoth. J., 46, 256-269.

Lollino G., Arattano M. and Cuccureddu M. (2002). The use of the automatic inclinometric system for landslide early warning: the case of Cabella Ligure (North-Western Italy). Pysics and Chemistry of the Earth, 27, 1545-1550.

Luzi G. (2010). Ground based SAR interferometry: a novel tool for Geoscience. In: Geoscience and Remote Sensing, New Achievements, edited by: P.Imperatore & D. Riccio, ISBN: 978-953-7619-97-8, 26 p.

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Nordvik T., Blikra L.H., Nyrnes E. and Derron m.-H. (2010). Statistical analysis of seasonal displacements at the Norndes rockslide, northern Norway, Engineering Geology, 114, 228-237.

Norland R. (2006). Differential Interferometric Radar for Mountain Rock Slide Hazard Monitoring. Technical report, 4p. Available on the Åknes/Tafjord Beredskap IKS website: http://www.aknes-tafjord.no.

ANNEX C

LANDSLIDE MITIGATION

SELECTED NATIONAL PERSPECTIVES AND EXPERIENCE

ROMANIA

SLOVENIA

SWITZERLAND

ROMANIA

C.1 ROMANIAN APPROACH TO LANDSLIDE HAZARD AND RISK MITIGATION MEASURES

C.1.1 OVERVIEW OF LANDSLIDE HAZARD IN ROMANIA

Every year landslide activity causes significant economic loss as well as loss of human life. In the view of experts from Central and eastern Europe (PECO) countries, in Romania landslides represent a high risk (**Figure C.1.1**),

In the rural environment, and particularly in mountainous areas, landslides represent a critical hazard. The total estimated area of landslides in Romania covers about 900,000 ha, putting at risk 60,000 households, 350,000 people, agricultural land, public and private buildings, utility networks and roads.

The areas of the highest landslide risk are located in the South Western portion of the Carpathian Mountains. The risk is mainly related to precipitation, slope angle, soil condition, land use and management.

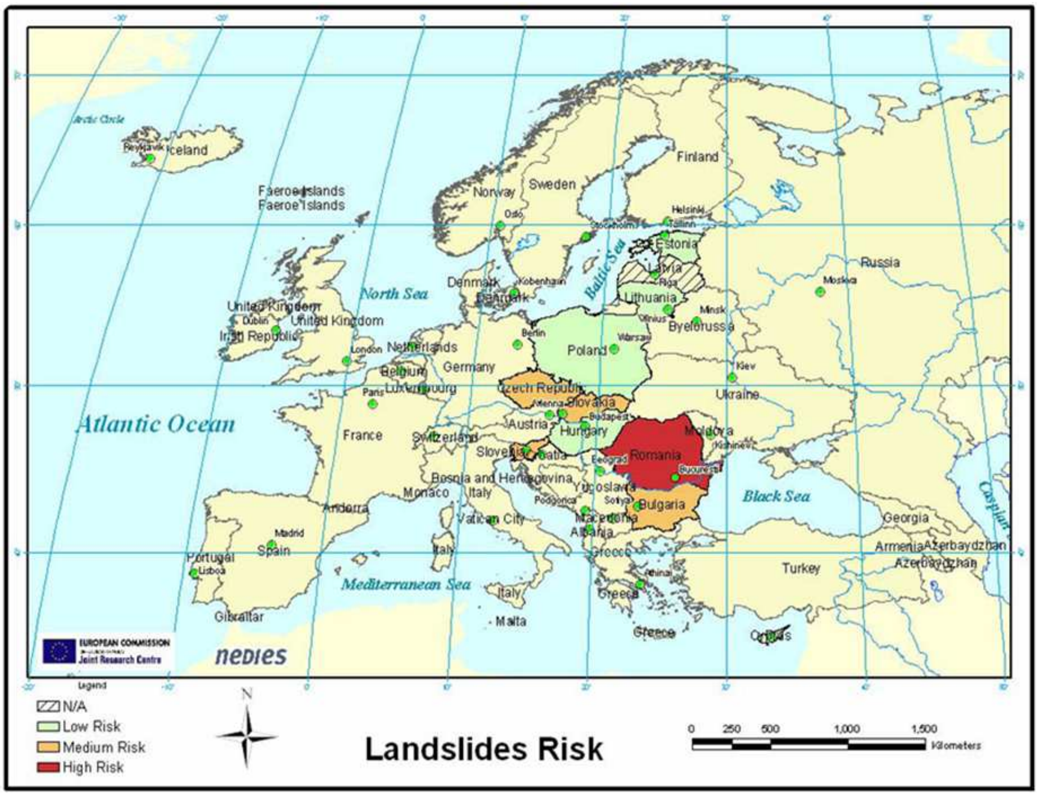


Figure C.1.1: Landslide risk in PECO countries

In Romania, landslides are closely related to floods and earthquakes. However, the studies and reports carried out by the Romanian Governement with the help of ONU in 2008 have shown that the role of the anthropic action is comparable with that of natural processes in elevating the risk.

Severe soil erosion, gullying processes, landslides and mud flows affects 30-40% of the total agricultural land. Landslides triggered by heavy rainfall and earthquakes usually affect the villages located on the slopes, while floods are the major risk factor for the network of settlements and lifelines along the main rivers.

Landslides play a significant role in the evolution of intra-and extra-Carpathian landscape, hilly regions and highlands which consist of flysch. The extension of these processes during periods of heavy rain, such as those that happened between 1969 and 1975, affected more than 11,000 ha in some counties such as Vaslui, Iasi, Mehedinti, Gorj, Valcea, Vrancea. In 1970 alone, a year characterized by extremely heavy rainfall, landslides affected 20 000 ha, comprising large areas of agricultural land, destroying many buildings and communication routes. Nowadays things are even worse: the number of affected terrains has grown exponentially as shown in **Table C.1.1.**

Table C.1.1: Adapted from "Report on the status of environment in Romania", 2009

Name	**Description**	**Surface (Ha) and the degree of affectation**					
		Weak	**Moderate**	**Strong**	**Heavy**	**Excessive**	**Totaly**
Soil erosion and landslides	Surface and depth soil erosion	602778.85	317546.44	224134.12	183976.3	147201.16	1449198.14
	Landslide	120070,53	525800,77	451501,63	173061,93	76899,45	1373772,71
Total		722849.38	843347.21	675635.75	357038.23	224100.61	2822970.85

C.1.2 ROMANIAN POLICY AND PRACTICE IN DISASTER MANAGEMENT

Romania is exposed to a range of natural disasters, particularly to the risk of earthquakes, floods, and landslides causing economic and human losses across the country. The expected annual property loss from earthquakes and floods is estimated at around 500 million euro. Since 1908, 14 earthquakes of magnitude VII or greater and 8 major floods were recorded affecting almost 2 million people and causing massive economic losses.

Romanian national policy in disaster risk reduction field is currently expressed through various legislative documents for the whole field and different risk types, administrative authorities, public institutions and specialized institutions with responsibilities in disaster prevention and response management.

The relevant laws regarding the national policy for disaster management are:

- Government Ordinance (GO) no. 47/1994, regarding the defense against disasters, approved by Law no.124/15.12.1995;
- Law no.106/25.09.1996 – Civil Protection Law, modified by G.O. no. 021/15.04.2004 regarding the National System for Emergency Situations Management.

At national level the system for the management of emergency situations is currently under reorganization, involving the redefinition of all the responsibilities for the national and local institutions playing a role in this field. According to the new laws currently under development, new institutions and operational structures will be created which will ensure that during emergencies people infrastructure and the environment will receive protection in a coordinated and professional manner.

C.1.3 RISK ASSESSMENT

Risk analysis must be based upon:

a) Hazard maps – geological, climatic, hydro -geological, technological, environmental; the maps will also indicate the zones of influence.
b) Maps of the risk elements.
c) Vulnerability maps.
d) Risk maps (direct and indirect losses).
e) Assessment of the accepted risk.

C.1.3.1 Hazards mapping and evaluation

The institutions in charge of detailing the methods to be used in Romania for evaluations of risks, vulnerability and response capacity are:

- National Institute of Research-Development for Environment Protection (ICIM) Bucharest;
- National Institute of Research-Development for Industrial Ecology (ECOIND);
- The National Institute for Building Research (INCERC);
- National Institute of Research-Development for Earth Physics (INCFDP);
- Institute of Nuclear Physics and Engineering “Horia Hulubei” (IFIN HH) Magurele;
- Institute of Geography form Romanian Academy;
- National Administration of Meteorology;
- National Institute of Hydrology (INH);
- Army Centre of Study and Research and Centre of Studies, Experiments and Specialization in prevention and fire fighting.

In accordance with the Law which in 2001 approved the Plan for National Territory Arrangement in Romania, maps have been developed for the risks of floods, landslides and earthquakes for every locality within the natural risk areas, containing information about the hazards, existing elements at risk and population, as well as the preventive measures applied. These maps will be included in the Plans for General Urbanism in order to implement the specific measures for building and terrain use, together with the definition of who will have free access to them.

Moreover, the maps and tables annexed to Law no 575/2001 provide information about the localities potentially affected by floods caused by river draining or overflowing and landslides. Maps of the biggest hydro technical dams were also developed.

Various research institutes and private companies developed electronic maps of risk. The Geographic institute of the Romanian Academy developed the map of 15 geomorphological risks (landslides, avalanches, erosion, etc) as well as numerous atlases referring to the natural and technological risks specific to the Romanian National Territory.

Detailed electronic maps of risks are developed for the administrations involved, for all types of natural and technological hazards, in order to support the territory development, to be used in the framework of defense plans in case of disaster.

Regarding the planning for general urbanism, which is defined at local administration level, the development of risks maps is currently under way at scales between 1:5.000 and 1:500, depending on the extension of the area of interest.

Within the plans for disaster response, risk maps are available at scales between 1:15.000 and 1:50.000 for local level, and 1:1.000.000, 1.500.000 and 1:200.000 at national levels. In this respect, GIS ArcView electronic maps were prepared at scale 1:1.000.000 for major hydro technical dams, chemical accidents, pollutions with hydrocarbons, nuclear accident and explosions. The maps adopted for disaster defense plans are in the GAUSS-Kruger coordinate system.

Furthermore, risks maps are employed by the major energy producers and transportation, construction companies etc. for information to the public.

At present, the responsibilities for risk mapping are shared among several commissions specialized on each class of disaster; those commissions are formed according to the Government Order no.47/1994. The maps are generally available to the public.

C.1.3.2 Vulnerability and capability assessment

By request of the operational team of Disaster Prevention and Preparedness Initiative, a National Plan for Disaster Management (NPDM in the following) in Romania was developed in 2001. Within this rather complex document, developed in collaboration by all the relevant institutions, the impact, intensity and evolution in time of the main types of hazards and vulnerability were assessed, as well as the human, material and financial resources available for hazard management.

During the assessment phase, the following were taken into account:

- the infrastructure elements (streets, bridges, buildings, etc) which could be affected by future disasters;
- the most vulnerable targets and the elements leading to their weakness with respect to each hazard class;
- increasing/decreasing number of vulnerable communities;
- the preparedness level regarding the risk factors at community level;
- the interest of communities in these issues, etc.

Moreover, the NPDM includes an analysis of the governmental and non-governmental bureaus involved in disaster management, the international cooperation in disaster situations as well as the capacities and challenges in disaster, prevention and preparedness, the gaps, imperative needs and demands in disaster management, at national and regional level.

The information provided by Romania through the NPDM, which was transmitted to the South-East Europe Stability Pact, contributed to the development of the “Gorizia” Regional Report on the Disaster Preparedness and Prevention Initiative..

Regarding the assessment of the vulnerability of the environmental factors with respect to the impact of economic and industrial activities, there are legal methodologies established by the Ministry of Environment and Waters Administration. On the basis of those methodologies, studies for the impact on the environment and functioning authorizations were issued. These studies are compulsory for all the economical agents and are developed by institutes and firms authorized and accepted by the Ministry of Environment and Waters Administration.

Regarding the evaluation of the vulnerability of industrial facilities, the methods applied are of two types: qualitative (HAZOP) and quantitative (HAZAN). The framing of industrial facilities and targets in categories of risks is defined based on these evaluations and the measures for reducing risks subsequently applied accordingly in order to decrease the vulnerability.

Two of these methods for the evaluation of risks and vulnerabilities for industrial objectives were finalized through projects of co-financing conducted in partnership with the Italian Ministry of Environment al Territory Arrangement: REHRA (with reference to the impact on waters) and TEIAMM (currently under way, which treats the aspects related to impact on air). All these methods and studies are conducted within ISO quality and efficiency standards provisions and are in accordance with the applicable European Norms.

C.1.3.3 Systems for Post-disaster impact assessment of the socio-economical and environmental damages

After each disaster, a systematic analysis of socio-economical and environment losses and impact is conducted, along with the definition of the disaster effects mitigation measures adopted, together with the measures that will be established to prevent that kind of situations. The results of the previous activities, presented to the Government and the public through mass media, can be examined by every interested person or institution.

The physical preliminary evaluation and the value disaster effects evaluation are a permanent care of the Romanian institution for the defense of the territory against disasters, aiming to realize some urgent operative measures and also medium and long term rehabilitation and reconstruction measures finalized to the normalization of the social-economic activities, and to promote the long lasting objectives.

At local level there are Commissions devoted to evaluation of consequences that use a specific methodology for the estimation of losses, to ensure compensations and to provide the necessary funds for situation normalization.

In case of disasters with major consequences, governmental commissions are responsible for the damage assessment, sometimes with the cooperation of international experts.

C.1.4 CLASSIFICATION OF MITIGATION MEASURES

The term "mitigation" refers to the actions which are put into practice to reduce the risk of damage and casualties. Mitigation can be conducted either by:

- Structural mitigation, which refers to any physical construction to reduce or avoid possible impacts of hazards, this includes engineering measures and construction of hazard-resistant and protective structures and infrastructure.
- Non-structural mitigation, which refers to policies, awareness, knowledge development, public commitment, and methods and operating practices, including participatory mechanisms and the provision of information, which can reduce risk with related impacts.

C.1.4.1 Structural mitigation measures

Structural damage, collapse of buildings or infrastructure are common consequences of disasters, including earthquakes, floods, and landslides. Structural mitigation aims to reduce this damage and to save lives through the reduction of the hazard and/or the reduction of the physical vulnerability of exposed elemnts.

Structural mitigation requires the expertise of civil engineers, including (a) the design of new buildings, roads, canals, dams, and other infrastructures, and (b) the strengthening and retrofitting of old structures. It is most important to ensure good maintenance of structures: if not accomplished, the poor quality of structures is often the cause of indirect damage.

Landslides can be triggered by many often concomitant causes and the reader is referred to other Safeland Deliverables (as D1.1) for a complete description of the factors which may induce a slope instability. With particular reference to Romania, in addition to shallow erosion or reduction of shear strength caused by seasonal rainfall, landslides are often triggered by anthropic activities such as adding excessive weight above the slope, digging at mid-slope or at the foot of the slope. Often, individual phenomena join together to generate instability, also after some time has elapsed, causing difficulties in the back-analysis of the landslide. This precludes a detailed reconstruction of the evolution of the landslide, other than in well-instrumented limited areas.

Details and examples of structural mitigation measures actually used in Romania are discussed in Section C1.6. Referring the reader to other part of Deliverable 5.1, devoted to the compendium of mitigation measures, it is worth mentioning here that in Romanian practice slope stabilisation methods are classified in the following three categories:

- Geometric methods, in which the geometry of the hillside is modified (usually the slope);
- Hydrogeological methods, in which an attempt is made to generally lower the groundwater level, or to reduce the water content of the material;
- Chemical and mechanical methods, in which attempts are made to increase the shear strength of the unstable mass or to introduce active external reactions to movements in order to contrast the destabilising forces; the reactions can be active (e.g. anchors, rock or ground nailing) or passive (e.g. structural wells, piles or reinforced ground).

C.1.5.1 Non-structural mitigation measures

Many types of non-structural mitigation measures can be simple and quick to apply and generally very cost-effective in reducing risk. Examples include land-use plans which define where human settlements and activities can be located or regulations that dictate which activities can or cannot be undertaken, depending on certain critical indicators; for example, early warning systems can temporarily restrict people from entering areas when the risk is above an admissible level. In addition to regulations and planning requirements, non-structural mitigation also refers to training people to recognize hazards to limit their their own exposure.

Our "cohabitation" with the hazard modifies the reference points of equilibrium, compelling us to a receptive and anticipative permanent and dynamic action, for example by:

1. Disseminating the new concepts of reducing the risk in case of disaster through the educational system. Both educational systems – the general culture system and the specialty system – should assimilate and disseminate disaster knowledge by a permanent transfer of information from researchers, practitioners and officials to the community.
   The education on reducing the risks in case of disaster will have to be a component of the development program, by organizing well-informed groups with an educational role at various levels:
   - political level (national planners, management administrators);
   - community level (community leaders, public, teachers, students, local civil and religious leaders);
   - voluntary level (voluntaries in case of disaster, spontaneous leaders).
2. Activating all the educational components by intertwining the formal education (school) with non -formal (extracurricular) and informal (direct experience) education.
3. Developing special educational programs of behaviour sociology
4. It was noticed that, in case of disaster, the people's behaviour differs according to race, ethnic group, religion, education (in the community, at school and in the family). The reaction to a disaster depends on the development of the human feeling of belonging to an habitat. The idea that a human being is cohabiting with hazard should turn from an attitude of resignation into one of involvement. This change in the human attitude is only possible through education.
5. Developing specific university specializations regarding risk management in case of disaster, by architecture and city planning strategies with the following structure:
   - Dissemination of specific scientific terminology (hazard at source, hazard at emplacement, elements exposed to hazard, vulnerability, risk) in order to favour its correct use by all those involved in decision and information and to overcome the current confusion and superposition of meanings attached to the different terms.
   - Introducing some new concepts which can foster multidisciplinary relations between different scientific departments – such as the principle of ecosystem approach, also applied in the constructed environment. This perception of the space organization establishes a hierarchy of the relationship of the individual -

collectivity (the anthropogenesis) with the environment (the biotope), which creates behavioral reference points that are fundamentally necessary to reconstruct in case a disaster occurs.

- Developing a relation between the university discipline and history of landslide in a given area, with that of management of risk reduction by implementation of a strategic system of global risk protection, by developing the concept of a security habitat.

6. The territorial planning within the limits of an accepted insurance percentage in case of disaster. It is necessary that every single village, town and city, especially those rated with “high’ and “very high” risk levels, have zoning maps that should take into account the implications of the risk over the planned development. The zoning maps must indicate the planning for different building categories (residential, social, industrial), the reserved zones and the special zones (for special risk buildings).

C.1.6 LANDSLIDE HAZARD MITIGATION MEASURES USED IN ROMANIA

C.1.6.1 General

In Romania, the most common causes of slope instability are excavations on the slopes, especially at their base, changes to the groundwater regime and deforestation. There are numerous examples showing that often construction works are designed and implemented without careful evaluation of all the implications that these works have on the environment, causing or aggravating instability. Subsequent restoration of damaged constructions required greater financial, material and additional human efforts than would have been required to prevent adverse effects in the first place.

There are frequent (and increasing) cases in which landslides require the adoption of complex solutions for their stabilization. Depending on the importance of the economic and social elements at risk, once landsliding has initiated the need to prevent further damage may call for immediate intervention with emergency works, which is often very expensive.

Ensuring the stability of slopes by acting on the disturbances factors is the most important aim sought by the designer. Its accomplishment requires the application of measures capable of preventing the resistance to be exceeded and contribute to its functionality. Consequently, the mitigation of landslides affecting important objectives involves the application of special measures and works to achieve a state of resistance consistent with the demand. **Table C.1.2** summarizes the specific categories and types of works normally adopted or considered for the mitigation of landslide risk in Romania; due to the high cost and difficulties of implementation, only some of the works listed in **Table C.1.2** remain as possible solutions to implement.

Table C.1.2: Special works to stabilize landslides commonly used in Romania

Special work categories	Type of work		Features
Work to improve resistance of slipped soil	1	Drainage Tunnels	Built both in water-bearing strata and beneath the sliding surface; the alignment of the drainage galleries follows influx of water or it assumes vertical wells
	2	Horizontal drainage	Kerisel Caqout-type drilling; in deep, thin aquifers mechanical vibrodrilling is used; can be combined with Benotto columns carried out in open drains or electro osmosis
	3	Vertical drainage wells	Used to relieve the pressure of the permeable layers; water is extracted by draining trenches
	4	Drainage trenches	Installed from the hill into the valley, by mechanized techniques, reaching bedrock; distance between draining trenches should not exceed 20 m; filter and drainage material occupies at least 1/4 of the trenches section
	5	Drainage by electroosmosis	The anode electrodes used are from steel tube and cathode electrodes from perforated pipe; DC source U = 50-150V, I = 25 A. The water is discharged by pumping
	6	Electrochemical consolidation	The electrodes used are from aluminum steel or calcium bars (anode), and copper (cathode) and are supplied with continuous current; at the anode clays are desiccated with H ions and on the cathode appear Al (OH) 2 and Fe (OH) 3
	7	Thermal treatment	Heating soil to temperatures of 500-800°C by burning a fuel (wood, coal, diesel); additional wells are needed to achieve a strong circulation in the cavity combustion
	8	Piles of soil stabilized with lime or cement	Run drills with Φ = 10-60 cm at depths of 10-20 m in the massive sliding. Place the soil mixed with lime and cement
	9	Treatment with surface-active substances and macromolecular polymers	Treatment of cohesive soils with vinisol, dialkildimethylammonium chloride, resins and epoxides etc..
Slope stability works	10	Slope Reprofiling and embankments	Excavation work to balance the masses of earth slopes by reducing slope angle. The aim is to reduce slip forces, increasing the resistance to sliding.
	11	Retaining and anchoring works (retaining walls, caissons, piles, bars, columns, piling)	For restoring stability of slipped soil . Continuous work on the entire front to be supported. Discontinuous work are made within the sliding mass

C.1.6.2 Special works to improve underground drainage

Drainage is used both to remove excess water and to modify the current underground hydrodynamic regime in periods of high groundwater levels or flows. Intervention is necessary in view of the fact that the cover layer accumulaties water in a fractured porous medium and to deal with very heterogeneous, but always present, hydrodynamic relationship between the different hydrogeological units in the slope (Livet, 1976).

For all types of drains, drainage has a positive effect on slope stability by increasing normal stress and thus shear resistance in the soil and by removing hydrostatic driving forces in tension cracks. When not drained, deep underground water filtration pressures can participate with 20-24% of the total shearing forces that contribute to rock (Coates and Brown, 1961). The disadvantage is that the underground drainage can be designed only after a detailed hydrogeological research of the slipped alluvium is conducted, so it shall be included within the category of long term works.

The main types of drainage work used in Romania to stabilize active landslides are (**Figures C.1.2, C.1.3, C.1.4)**:

a) drainage galleries are recommended for deep landslides, which have a large amount of water to be discharged. They can be drilled just below the sliding surface: the upper layers of water collection is performed by means of installing vertical wells at the crown of the tunnel, essential where landslides deep drainage must be performed with tunnel length exceeding 200 m (Zaruba and Mencl, 1974).

b) Horizontal drilled drains have wide application as a landslide stabilization technique. They can be set up by means of helical drills, mills, roller or vibro drilling, installed from excavations or circular caissons. The process involves drilling holes of 20 to 200 m long with a slope angle of 3-10° and a diameter of 65-90 mm. The casing pipes used in association with rotating drill bits or rollers must be thick-walled tubes, to avoid torsional deformation; they serve as support during drilling, which is done in the presence of drilling mud. Vibro-drilling has become accepted as a suitable technology for the installation of horizontal drainage due to lower cost per unit length, the greater speed of execution and the reduce labor. The installation phase requires the installing a reinforced concrete caisson with a diameter of 3 m, fitting the vibro instalation equipment ath the base of the caisson and the actual drilling. After the completion of drilling, a perforated PVC tube is inserted. The installation of a drain 25 m long and 100-140 mm thick requires approximately 1 - 1.5 hours.

c) Vertical drainage wells are characterized by a minor application in landslides stabilization work. They have been used to discharge in deep layers water abstracted from coastal springs (Tarina Valley, Perieni, Vaslui County)

d) Draining trenches with depths up to 10 to 12 m are used routinely to stabilize landslides with large and medium depth. The design of trenches demands for a thorough knowledge of the geological and hydrogeological conditions, in order not to affect the stability of the slope. From the point of view of the construction, they can be executed along the line of greatest slope angle, in the form of arches and along contours, with mechanical technologies.

(a) (b)

Figure C.1.2: DJ10 (County road)- affected by landslide in Chiliile , Buzau County November 2009. Earth embankment works. (a) Filling the dig for the surface drains; (b) Filling the dig without the surface drain.

Figure C.1.3: Houses of Chiliile village, Buzau County, affected by landslide in March 2010 – draining work in progress

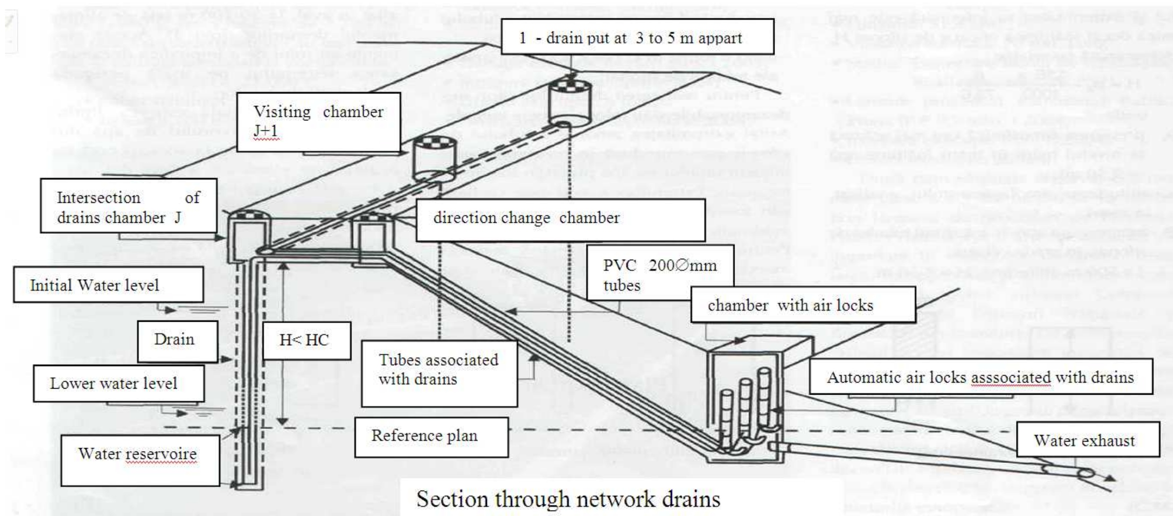


Figure C.1.4: Section through network drains

C.1.6.3 Special measures to increase the resistance of landslides

In special circumstances, due to the urgency of adopting measures to stabilize a slope, several methods are available to enhance the resistance of soils. Among them the following are noted:

a) Drainage through electro-osmosis (electro-draining) was first used by Casagrande (1941) to stabilize the cut slopes of a norwegian railway line. Electro-osmosis has the same effect as underground drainage, but it differs in that the drainage water is moving under the action of an electric field. The method applies very well to clay and muddy-clays-rocks, but it becomes inefficient for fine sands. Electro-osmosis drainage consists in the introduction, into the sliding mass, of electrodes from steel tube-anode and perforated pipe (needle filters) for the cathode. The electrodes, located at a spacing of 3-10 m, are connected to a source of continuous current of 50-150 V and 25 A. The resulting electric field produces a shift of water from the anode (+) to cathode (-), where it is evacuated with needle vacuum filters.

b) Electrochemical strengthening field is achieved by placing the electrodes formed of aluminum or steel (anode) and copper (cathode), connected to a source of continuous current. The electrolysis process induced by the electrical potential difference leads to the decomposition of water by separating hydrogen and oxygen at the anode and at the cathode, anode decomposition of metallic cation and movement of soil solutions. It was applied with positive results in areas of several civil and industrial construction in Iasi, Braila, Galati, Navodari and some mining tunnels.

c) Thermal treatment is known from a long time but its use was limited due to demanding technologies and high energy consumption. In applying this method the structure of the mineralogical constituents is modified, leading to the calcification effect. Heating is achieved by diffusion, from the external heat source, either by direct combustion with injection wells or tunnels. To intensify the process of combustion and heat spreading effect in the soil, two communicating wells are required, with combustion achieved in one of the two.

d) Inclusions (columns) of soil stabilized with lime or cement. This applies to clay and consists of drilling with a diameter between 10 and 70 cm, down to depths of 10-12 m (see for example **Figure C.1.5**). Earth mixed with lime or cement is inserted into the drilled hole, thus resulting in a column of treated material which increase the soil mass resistance. Sometimes, after water extraction, cement concrete or mortar with additives is injected, controlling the pressure flow and setting time, and resulting in columns of reinforced material. These effects occur in the short term. However, there is a reduction in soil moisture around the drilling, with 0.6 to 0.8% for a dosage of 1% lime, caused by the water used for the hydration of calcium oxide.

e) Treatment with macromolecular surfactants and polymers. Increasing soil residual shear resistance along the sliding surface, represents nowadays the subject of recent research interest. Researchers are considering the creation of bulbs or blocks, under the pressure of injected stabilizing agents. The results obtained by introducing under pressure a solution of water, cement and soil material have been found as satisfactory only in unsaturated clays.

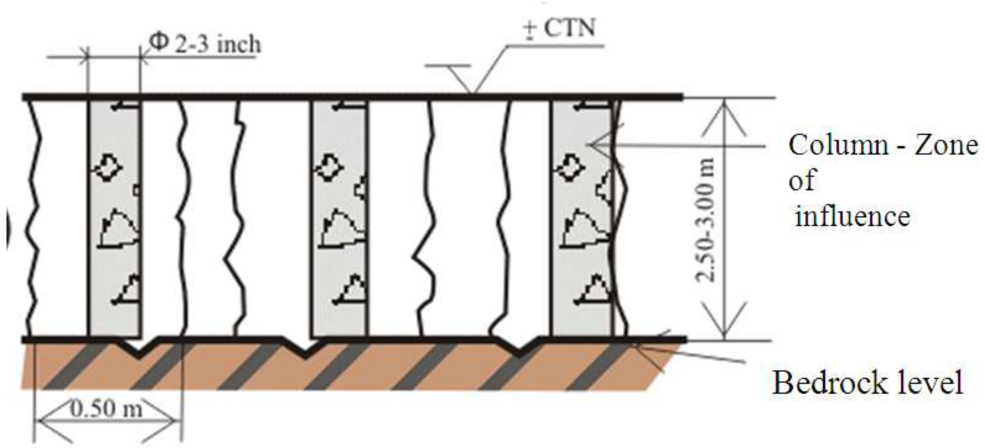


Figure C.1.5: Network of lime Colums (1-1.5 m distance between colums)

C.1.6.4 Special work to retain landslides

Construction works to retain landslides are designed to increase stability and protect the transport networks and existing buildings. In relation to the length required by the sliding front to be consolidated, and the forces generated by the soil mass, technical solutions are different and dependent on social and economic importance of the target. Some examples are illustrated in Figures **C.1.6**, **C.1.7** and **C.1.8**)

Soil renforcement by means of vertical elements (columns, drilled anchors, piles, etc..) is not new. A special case is constituted by soil reinforcement methods for running old solutions (obtaining resistant materials using straw mixed with clay land), based on the principle of placing in the ground synthetic textile materials to ensure stability and reduce deformation. Vidal's solution (1966) of reinforced earth was much improved by using geosynthetic materials, particularly to strengthen the main slopes and landslides regressive reconstruction of excavation slopes. Reasearch conducted on these topics show that earth reinforcement gives a certain rigidity reducing deformation. According to the results obtained by Saran et al. (1979), Petrik et al. (1982) and Christie (1982), the effect of reinforcement works are:

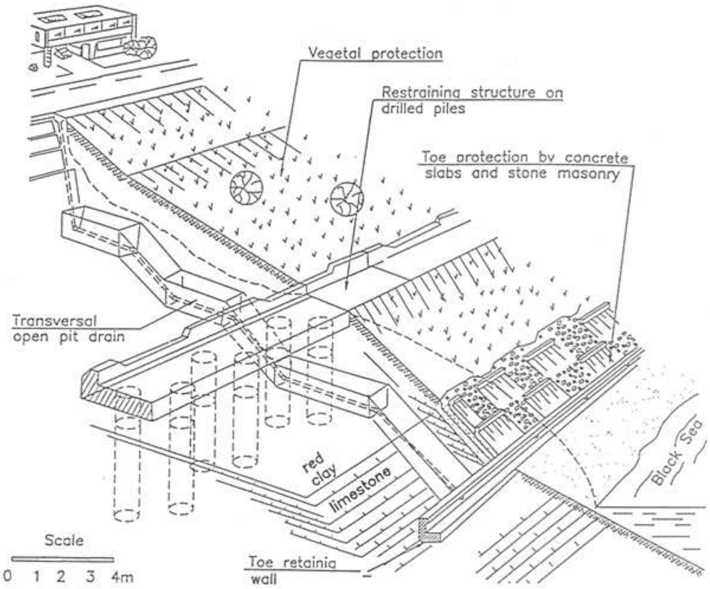
- Reduction of the sliding surface tension by 20% when using geotextile materials, and about 50% with steel bands.
- Reduction by about 30-40% of the load on earth works.
- 15 to 20% increase in overall soil stiffness.
- replacing, in some cases, of the traditional support structures.
- 40-50% reduction of expenditure on the classical embankments works.

Although not part of the group works to stabilize landslide, reinforced nets to protect land against erosion are adopted widely. Thanks to their special geometry, installed directly on the ground these nets prevent the displacement of soil particles, the formation of run-off and ravines, contributing substantially to reducing erosion. On the land protected by nets, vigorous vegetation usually develops, with plant roots protected and secured. Before and after installation it is recommended to spread topsoil on slopes mixed with perennial grass seed. In this manner the vegetal cover installation is much faster, resistant and dense.

Figure C.1.6: Retaining walls, gabions, terracing works, drainage, application of soil reinforcement solutions or afforestation. Works of "Stabil Ambient" in Romania, 2009.

Figure C.1.7: Example in Romania: torrent planning, repairs to retaining walls of routh stone, consolidations including nets anchored slopes, National road 7-Olt Valley, 2009

Figure C.1.8: National Road 10, Buzau County, February, 2010.



Remedial measures in progress:

- Modification of slope geometry by reducing general slope angle
- Limiting the unfavourable effect of precipitations/ groundwater by appropiate drainage systems : surface drains and shallow trench drains
- Planting the slope surface
- Preventing marine erosion by extending and upgrading the see defence works: retaining walls and protective concrete blocks
- Adding stabilizing force by passive piles
- Heat treatment.

Figure C.1.9: Complex stabilization works in Costantza city area, Romanian Black Sea shore (Popescu, 2002)

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SLOVENIA

C.2 GENERAL EXPERIENCE AND PRACTICE IN SLOVENIA

C.2.1 OVERVIEW OF LANDSLIDING IN SLOVENIA

The Republic of Slovenia, located in Central Europe between the Alps and Adriatic sea, became an independent and sovereign country on 25 June 1991. Slovenia (20.000 sqr. km) is positioned on the complex Adria – Dinaridic – Pannonian structural junction (**Figure C.2.1**). Although the general geological structure is well known, details may come as surprise. As a consequence of its geological setting, Slovenia is very much exposed to slope mass movement processes and almost one quarter of Slovenian territory is subjected to processes of soil and rock movements and, based on rough estimations, around 18 % of the population is under threat by these phenomena.

The Slovenian territory is, geologically speaking, very diverse and mainly composed of sediments or sedimentary rocks. Slope mass movements occur almost in all parts of the country. Rock falls, rock slides and even debris flows can be triggered in the Alpine carbonate areas of the northern part of Slovenia. In the Alps, rock slides and rock falls are frequent. For example, numerous rock falls and slides were observed in western Slovenia during large eartquakes in 1976, 1998 and 2004. Rockfalls are also present in those areas, where rivers have cut gorges through hard carbonaceous rocks into the lower-lying soft clastic sediments.

Landslides are present first of all on the hillsides and slopes of the perialpine terrain composed of carbonaceous and clastic rocks. Large landslides in such rock strata are frequent, where the thick weathered surface layer is sliding. Large soil landslides are quite usual in the mountainous regions of central Slovenia composed from different clastic rocks, while there is a large density of small soil landslides in the young soil sediments of the eastern part of Slovenia, where the hilly terrain, with relatively gentle slopes and wide valleys, is composed of clayey and silty soils / sediments, sometime marl, sand and clayey gravel. These hard soils/ soft rocks are subjected to strong weathering and form the basis for frequent soil slumps in thick weathered surface layers and along the inclined clayey layers. Landslide-safe areas in Slovenia are karts plateaus and karst heights, wide lowland basins and alluvial valleys.

Landsliding is not only a threath for buildings of any kind and to infrastructure in general but also changes the morphology of the terrain. Landslides often release (destabilize) large amounts of sediments, which not only stay on the slopes but also reach the fluvial network. Under catastrophic conditions, landsliding may lead to torrential outburst, debris flow, or dam-brake wave, as was the case in November 2000 with the first Stože debris landslide that turned after 35 hours into a deadly debris flow. Minor landslides in Slovenia are of different forms (mainly shallow landslides, with abundance of smaller slides and slumps). They are mainly triggerred durig short and intense rainfall events or after prolonged rainfall periods of moderate intensities. The order of their volume is 1,000m^3, rarely 10,000m^3. Some of them have already been stabilized using technical measures, others are still active.

C.2.2 LANDSLIDE HAZARD MAPPING AND DATABASES IN SLOVENIA

Unfavorable geological conditions are the main cause for such a high landslide density within Slovenia (>1 slide per 10km^2), despite good vegetation conditions. Such high slide density was confirmed in perialpine Slovenia using multivariate statistical methods. As a result of such an approach, a landslide susceptibility map of Slovenia was prepared (**Figure C.2.2**). The next contributing factor is the abundance of precipitation and high number of days with daily totals above 20mm. Many slumps and slides are triggered during short and intense rainfall events or afetr prolonged rainfall periods of moderate intensities. Slope creep is common in Tertiary over-consolidated clays and Permo-carboniferous claystones and shales.

Depending on yearly weather conditions, a few tens to a few hundred new instability phenomena emerge every year. Erosion appears nearly in half of the Slovene territory, mostly in connection with mountain torrents, where large uncovered rock areas are revealed. The consequence of landslides and erosion phenomena is the creation of unusable areas, becoming at the same time a thread to different objects and targets.

For the area of Slovenia, a debris-flow susceptibility map at scale1:250,000 was also produced (**Figure C.2.3**). Values in the legend indicate the susceptibility to debris-flow: 1 – insignificant; 2 – low; 3 – medium; 4 – high; 5 – very high. The grey areas represent areas where the debris-flow susceptibility is negligible. The results show that approximately 4 % of Slovenia is extremely susceptible and approximately 11 % of the country is highly susceptible to debris-flows. As expected, these areas are related to mountainous terrain in the NW and N of Slovenia.

The new rockfall susceptibility map is currently in progress: it will complement the set of susceptibility maps of different mass slope movements.

In Slovenia over 6,600 active and mainly minor landslides have been registred so far. Not all of them are part of the official landslide inventory that was incorporated into a GIS enviroment, using a software called GIS-UJME, developed mainly by the Ministry of Defense. The landslide inventory maps include more than 3,500 landslides, but not rock falls and rock slides. It is one of the 85 geo-referenced layers incorporated in the database, together with infrastructure, flood hazard maps, avalanche inventory, earthquake hazard maps, etc. This electronic database is used as an internet application by the Ministry of Defence in regional Notification Centers for coordination purposes during immeadiate disaster relief actions led by the Civil defense units, and as an intranet application for the training in the Protection and Rescue Education and Training Center and for the preparation of civil protection and disaster relief plans in the Administration of the Republic of Slovenia for Civil Protection and Disaster Relief. Unfortunately, this database is (still) not directly used for planning activities in the Ministry of the Enviroment and Spatial Planning for hazard prevention.

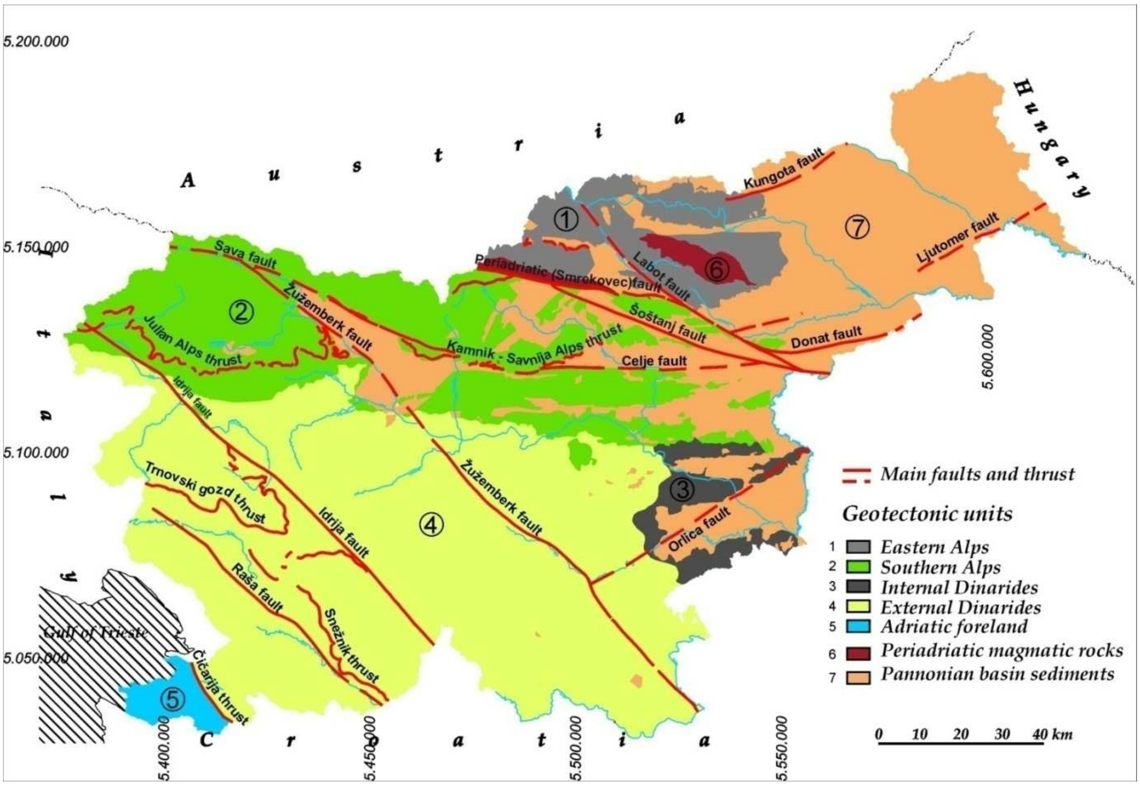

Figure C.2.1: Tectonic setting of Slovenia

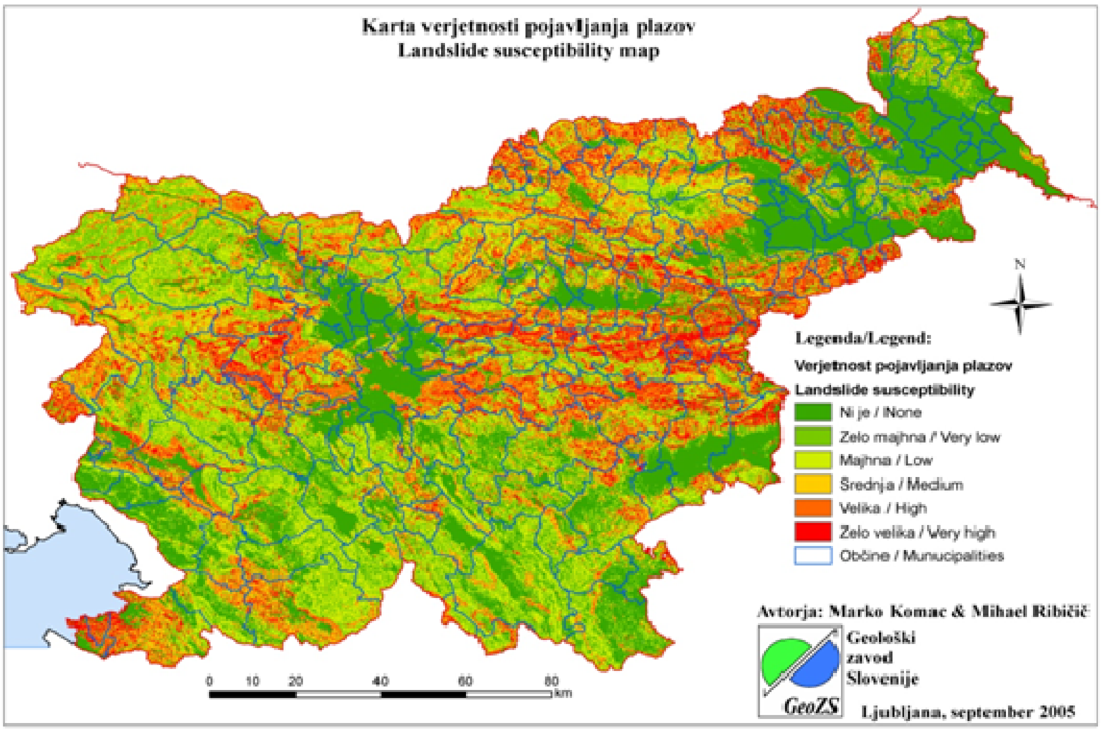

Figure C.2.2: Landslide susceptibility map of Slovenia

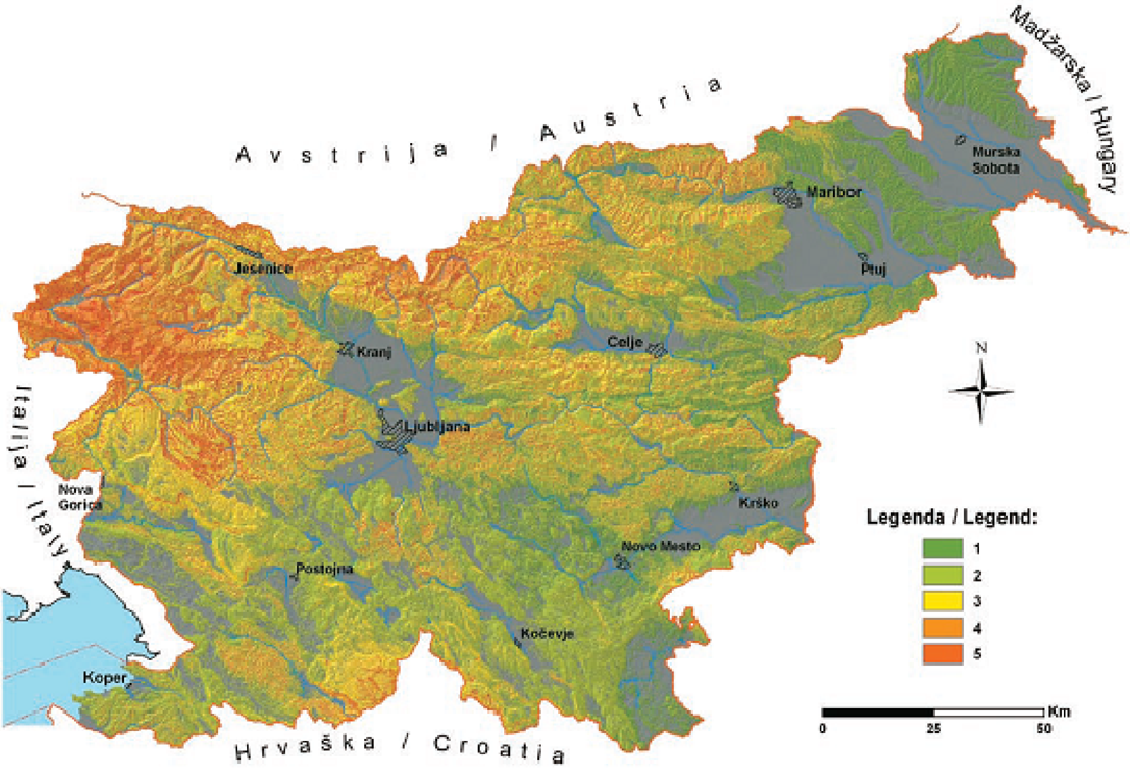

Figure C.2.3: Debris-flow susceptibility map

The damage caused by slope mass movements is high, but still no common strategy and regulations have been developed yet to tackle this unwanted event, especially from the aspect of prevention. One of the first steps towards an effective strategy of fighting against landslides and other slope mass movements is a central landslide database, where (ideally) all known landslide occurrences would be reported, and described in as much detail as possible. At the end of the project for the implementation of the National Landslide Database, May 2005, there were more than 6,600 registered landslides, of which almost half occurred at a known location and were accompanied by the main characteristic descriptions. The assessed database is a chance for Slovenia to start a solid slope mass movement prevention plan. The only part which is missing and which can be considered as the most important one is the adoption of a legal act that will legalise the obligation of reporting slope mass movement events to the authorities responsible for the database population.

Legislation, planning and prevention measures are not satisfying in the field of landslides and erosion processes in Slovenia. The legislation adopted in the last few years remains on general level (Environment Protection Act, Protection Against Natural and Other Disasters Act, Water Act, National Programme for the Protection Against Natural and other Disasters,) and does not demand making of instability risk maps that should be obligatory. The financial resources used for prevention measures are also much too low.

C.2.3 CURRENT STATUS AND RECOMMENDATIONS FOR LANDSLIDE RISK MANAGEMENT IN SLOVENIA

Damage caused by landslides and avalanches in Slovenia is large and it summed to 84.8 million EUR in the period of 1994 to 2003, not accounting for the remediation costs. As stated in the previous paragraph, a National landslide database exists (GIS_UJME), but it has not been operational since 2005. The landslide data are scattered among different institutions. The last evidence reported from the Slovenian municipalities in the year 2005 contains 1677 active landslides that pose a threat to the infrastructure and buildings. Depending on yearly weather conditions, a few tens to a few hundred new instability phenomena emerge every year.

The primary activities are still focused on the remediation instead on the prevention measures.With respect to damage prevention or its minimisation, a much higher focus on prevention would be logical and the only logical solution. More rigorous spatial planning restrictions should be imposed on areas susceptible to slope mass movements to prevent damage to objects, infrastructure, and soil. Susceptibility and geohazard maps based on knowledge and understanding of influential spatio-temporal factors affecting slope mass movements represent the basis for the sound spatial planning. Its maintenance and updating represent a valuable source of information for understanding slope mass movement occurrences, while susceptibility and geohazard maps represent one of the key information for sustainable spatial planning. Regarding the current situation, the first step ought to be reanimation of the landslide database and inclusion of geohazard and susceptibility maps in spatial planning processes.

Insufficient implementation of prevention measures results in damage occurring at times of extreme precipitation which can be several times greater than prevention investment cost. The fundamental prevention measures should be: (1) hazard and risk estimations, (2) avoiding new housing development on threaten areas and (3) preventive hazard mitigation (stopping the spreading of landslide and erosion areas, stabilization of the sliding surfaces and torrents).

C.2.4 MITIGATION MEASURES

Mitigation relates to concrete actions which are put into practice to reduce the risk of destruction and casualties. Mitigation could be generally divided into two main types of activities (see main text of the Deliverable for further discussion):

1) Structural mitigation refers to any physical construction to reduce hazard or to avoid or minimize possible impacts; this includes engineering measures and construction of hazard-resistant and protective structures and infrastructure. The following comments provide an overview of structural mitigation measures used in Slovenia to date. A detailed list is shown in **Table C.2.1**:
   - GeoZS creates its own landslide database with 803 records (included in GIS_UJMA).
   - Short statistics on these 803 landslides: 24% (193) are “manmade” (18,7% loading the head of slope ; 5,2% decreasing the toe of the slope ; 0,12% drawdown the

water table) and 76% (610) are “natural” (73,7% heavy rainfall and 2,2 % river bank erosion).

- In past years structural methods were prevailing, especially retaining structures with subsurface draining (gravity walls, anchored walls, cantilever walls, pile walls).
- Surface drainage for surface protection is common; drainage is generally the most cost-effective solution; high quality drainage system is needed!
- Several drainage system devices were used, depending on: slope geometry, ground material (soil or rock) and slope charachteristic (steepness, vegetation, ..)
- Measures for structural reinforcements and modification of material properties were rarely used
- On terrain susceptible to creep, trees and shrubs are often needed to decrease local instabilities

2) Non-structural mitigation refers to policies, awareness, knowledge development, public commitment, and methods and operating practices, including participatory mechanisms and the provision of information, which can reduce risk with related impacts. The following comments provide an overview of structural mitigation measures used in Slovenia to date:

- The financial resources used for prevention measures are much too low
- Due to updated law since 2007, land use planning must include possibility of floods and landslides
- Land susceptibility map (multivariate analysis of predisposal factors): since 2005.
- Hazard maps for municipalities (6 already prepared, 14 in progress; altogether 210 municipalities).
- Raising of public awareness: interviews on TV, articles, informative internet pages; Geological Survey of Slovenia(GeoZS) produced some informative »letters« for public which are available on our web site (http://www.geo-zs.si/podrocje.aspx)
- Insurance - people in Slovenia are not sufficient aware of it’s importance .

Table C.2.1: Structural landslide remedial measures used in Slovenia

No.	Description
1.	Modification of slope geometry
1.1	Removing material from the area driving the landslide (with possible substitution by lightweight fill).
1.2	Adding material to the area for maintaining stability (counterweight berm or fill)
1.3	Reducing general slope angle
2	Drainage
2.1	Surface drains to divert water from flowing onto the slide area (collecting ditches and pipes)
2.2	Shallow or deep trench drains with free-draining coarse granular fills and geosynthetics
2.3	Buttress counterforts of coarse-grained materials (hydrological effect)
2.4	Vertical (small diameter) boreholes with pumping or self draining
2.5	Vertical (large diameter) wells with gravity draining
2.6	Subhorizontal or subvertical boreholes
2.7	Drainage tunnels, galleries or adits
2.8	Vegetation planting (hydrological effect)
3	Retaining structures
3.1	Gravity retaining walls
3.2	Crib-block walls
3.3	Gabion walls
3.4	Passive piles, piers and caissons
3.5	Cast-in situ reinforced concrete walls
3.6	Reinforced soil structures with strip/ sheet - polymer/metallic reinforcement elements
3.7	Buttress counterforts of coarse-grained material (mechanical effect)
3.8	Retention nets for rock slope faces
3.9	Rockfall attenuation or stopping systems (rocktrap ditches, benches, fences and walls)
3.10	Protective rock/concrete blocks against erosion
4	Internal slope reinforcement
4.1	Rock bolts
4.2	Micropiles
4.3	Soil nailing
4.4	Anchors (prestressed or not)
4.5	Grouting
4.6	Stone or lime/cement columns
4.7	Vegetation planting (root strength mechanical effect)
5	Surface protection
5.1	Slope surface protection (used to reduce erosion and water infiltration)
5.2	Impermeable surface protection (sprayed concrete)
5.3	Biomeshes

C.2.3 USING NEURAL NETWORKS IN LANDSLIDE RISK ASSESSMENT IN SLOVENIA

There was interest in evaluating the suitability of neural networks for solving slope stability problems around roadways. In calculating stability of landslides, slopes and sideslopes, various analytical and numerical methods can be used. However, by means of such methods, one could not take into account a wide number of characteristics that influence slope stability. First, suitable input characteristics were chosen in order to predict slope stability in the research and assessment phase. Proper training of a neural network depends on the choice of input data, which include all geological possibilities for landslide appearance. A neural network can only assess properly in circumstances for which it was trained.

Among more than 100 project reports on landslides, road construction sites, new regional road construction and motorway construction, representative cases with an agreeable set of correct data were chosen. For each landslide, data on site and laboratory investigations had to be available. In data analysis, landslides on roads were chosen, being the procedure and determination of suitable input characteristic very time consuming.

Problem: choice of characteristics that influence landslide stability and are at the same time investigated and described in the majority of project reports. On the basis of project reports, 11 parameters or characteristics, which sufficiently describe a landslide, were determined.

The aim was to investigate the use a neural network to predict slope stability. Characteristic cross-sections were chosen in the landsliding areas. Each characteristic cross-section was described with the chosen parameters. Common characteristics of the studied landslides:

- They all cross a road.
- The study was limited to land/soil slides.
- Only cross-sections with adequate number of data were considered.

Each cross-section was described with the following parameters:

- Slope inclination: in degrees; important data as remediation is normally done with decreasing of slope inclination
- Bedrock inclination: in degrees; steep inclination is a potential sliding surface for the above landslide
- Bedrock type: water, which normally appears at the contact between landslide and bedrock soaks the bedrock and worsen geomechanic characteristics of the bedrock (class 1 – impermeable clastic rocks that soften when in contact with water; class 2 – carbonate rocks not considerably influenced by water)
- Depth to the bedrock: in meters; as relevant is given the largest distance to the bedrock
- Geology of the landslide: described is prevailing material with the poorest geomechanical characteristics (class 1 – coherent soils; class 2 – incoherent soils)
- Soil granulometry: a landslide consists of parts with good and poor geomechanical characteristics; for our purposes they were divided into five categories
- Landslide consistency (state of thickness): it is described for the soil with the poorest geomechanical characteristics, there are 4 categories for coherent and 4 categories for incoherent soils.
- Highest degree of moisture: soils with the highest % of moisture

- Smallest shear angle of landslide: determined with reversible stability analysis or geomechanical laboratory tests
- Type of other matrix in a landslide: for now the described characteristics describe parts of a landslide with the poorest geomechanical characteristics. As landsliding depends also on geology of other parts of a landslide we added another characteristic that describe a type of soil that constitutes other parts of a landslide: class 1 – coherent soils; class 2 – incoherent soils
- Water appearance: categorisation into five categories.

According to the described methodology, 95 cross-sections on landslides and stable slopes were listed, of which there were 53 landslides and 42 stable slopes. Among the listed cross-sections, 67 were chosen to constitute a training file and 28 to form a test file. The chosen slopes are covered with fine-grained and course-grained material. Chosen are cases with different bedrock: flysch, marl, grey clay, dolomite, limestone, claystone, sandstone and diabase.

For slope stability assessment, three types of neural networks were used:

- back-propagation: the best results
- learning vector quantization – LVQ
- self-organizing maps

The results of slope stability calculation with all three types of neural networks showed that slope stability can be predicted on the basis of 11 input characteristics. A neural network assesses slope stability with 96 % accuracy. Among all 11 characteristics, shear angle and moisture percentage are the most difficult to determine. They are measured in geomechanical laboratory tests, but they are often unavailable in the first analysis and assessment. The neural network learned properly to distinguish between stable and unstable slopes even if shear angle and moisture percentage were not used in the training process. In this case the neural network accurately predicted slope stability in 89 %, which is still a high percentage.

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SWITZERLAND

C.3 EXAMPLES OF PROTECTION MEASURES IN SWITZERLAND

C.3.1 INTRODUCTION

Natural hazards include all processes and impacts of nature which can be damaging to human beings and material assets. Natural disasters of a catastrophic extent have always occurred in Switzerland. However, as housing schemes have become denser and material assets bigger and more valuable, the scale of damage has considerably increased over recent decades. Switzerland is frequently affected by floods, storms, landslides and avalanches. Less frequent hazards include droughts and heat or cold waves. Strong earthquakes are very rare, but they can occur, as history has proved (www.planat.ch).

Landslide mitigation works are conducted in order to stop or reduce landslide movement so that the resulting damages can be minimised. Landslide mitigation works are broadly classified into two categories: 1) control works; and 2) restraint works. The control works involve modifications of the natural conditions of landslides such as topography, geology, groundwater, and other conditions that indirectly control portions of the entire landslide movement. The restraint works rely directly on the construction of structural elements. Specific measures included in the control works and restraint works are listed in **Table C.3.1**.

Four case histories from Switzerland are described below:
1. Stabilisation of the Toggenburg rock slope;
2. Deep Drainage of the Campo Vallemaggia landslide;
3. Pontresina Check Dam;
4. Arschella Ost creeping landslide.

The natural hazard protection measures adopted are (**Figure C.3.1**):
- Reducing risk from rock fall hazard (1);
- Drainage of the water (2);
- Stopping, guiding, draining debris flows (3);
- Decelerating creep movements (1, 2 & 4)

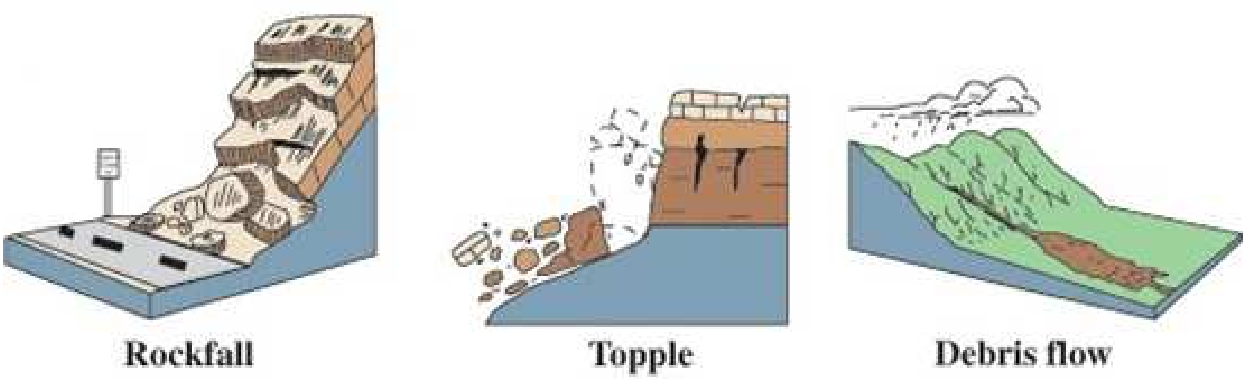


Figure C.3.1: Schematic representation of the protection measures adopted in Switzerland (adapted by Springman et al 2011).

Table C.3.1: Classification of landslide mitigation method
(after http://www.tuat.ac.jp/~sabo/lj/ljap4.htm, adapted by Springman et al., 2011)

Category	Method	Treatment
Control works $E_d \leq R_d$	Surface drainage to reduce water infiltration	Seepage barrier, surface drains, drainage blanket, capillary barrier
	Sub-surface drainage to remove the ground water within or to prevent water from flowing into the landslide mass	Shallow: horizontal drains, trench drains; Deep: deep wells, well point and ejector systems, relief wells, vertical gravity drains, tunnels and drainage adits, vertical shaft with drainage array
	Soil treatment	Electro-osmosis, vacuum dewatering
	Soil removal	Weight reduction or regarding
	Soil fill	Buttress or toe berm
	Erosion control	Stabilization/protection of river banks
Restraint works $E_d \leq R_d + \Delta R_d$	Sheer piles	Driven piles, steel piles, large size cast-in-place piles
	Anchors	Soil nails and anchors
	Retaining walls	Crib, gravity, tieback, sheet pile, soldier pile
	Earth reinforcement	Mechanically stabilized soil
	Biological stabilization	Vegetation for stabilization or protection
	Slip surface strenthening	Grouting
NOTE E_d are the actions at design level, R_d are the complementary resistances in the ground and ΔR_d the additional stabilising measures		

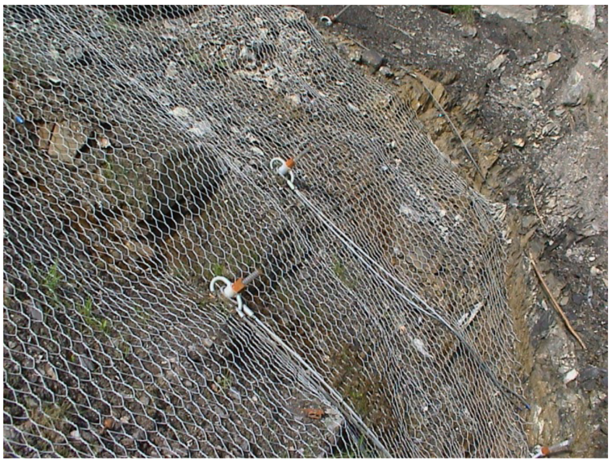
C.3.2 STABILISATION OF THE TOGGENBURG ROCK SLOPE

Toggenburg is the name given to the upper valley of the Thur River, in the Swiss Canton of St. Gallen. The valley descends in a northwestern direction from the watershed between the Rhine and the Thur, and is enclosed on the northeast by the Säntis mountain range and on the southwest by those of the Churfirsten and the Speer. The mitigation measures to prevent weathering and erosion of the steepest slope are: anchored concrete beams (with load cells to monitor the pre-stress applied); grouted nails, nets and greening, as reported in **Figure C.3.2**.

(a)

(b)

(c)



(d)

Figure C.3.2: - Mitigation measures adopted for the Toggenburg rockslope: (a) netting ; (b) anchor concrete beam with load cell; (c) steel nails; (d) net with nails (Photographs: Springman). The classical treatments in Switzerland often include a combination of pre-stressed anchors to secure the deepest unstable zones, and grouted nails to stabilise the potential shallower instabilities. These methods are often used together. Protection against any possible corrosion is fundamental for both types of long term structural measures.

C.3.3 DEEP DRAINAGE OF THE CAMPO VALLEMAGGIA LANDSLIDE

The Campo Vallemaggia creeping landslide is located in the crystalline penninic nappes of Ticino, in southern Switzerland, 50 km NW of Lugano. Two small villages, Campo Vallemaggia and Cimalmotto, are located on the toe of the slide mass, and surface displacements have been geodetically measured for over 100 years. Surface and borehole investigations of the Campo Vallemaggia landslide have shown that the unstable mass incorporates approximately 800,000,000 m^3 of weathered and intact rock (Bonzanigo et al., 2007). Surface and borehole investigations of the unstable mass suggest that the yield and sliding surface (actually a zone several metres thick) reaches a depth of up 300 m. A schematic representation of the region, and the geologist's block model, are reported in **Figure C.3.3** (after Bonzanigo et al., 2007; Eberhardt et al. 2007).

The measure adopted to mitigate against the deep seated creeping landslide is a drainage tunnel, as shown in **Figure C.3.4**. The water table has been successfully drawn down with considerable settlements developing during this period (prior the mitigation v = 5 cm/year with an average of 30 cm/year over the past 100 years due to several short periods of acceleration). The slope has virtually stopped creeping.

Geodetically measured slope movements were seen to decrease significantly across the entire slide mass, and in some cases, upslope displacements were recorded relating to the development of a subsidence cone. Surface geodetic measurements revealed that up to 40 cm of vertical consolidation subsidence occurred directly over the drainage adit (**Figure C.3.4**c). Given the kinematic constraints imposed on the Cimalmotto block by the Campo block (Bonzanigo et al. 2007) the stabilization of the Campo block had a similar stabilizing effect on the Cimalmotto block.

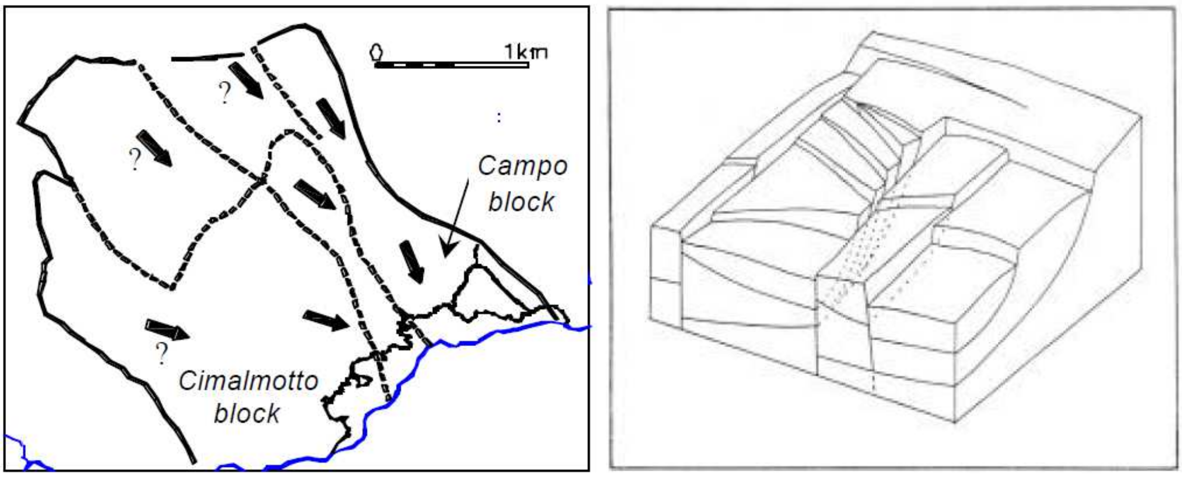
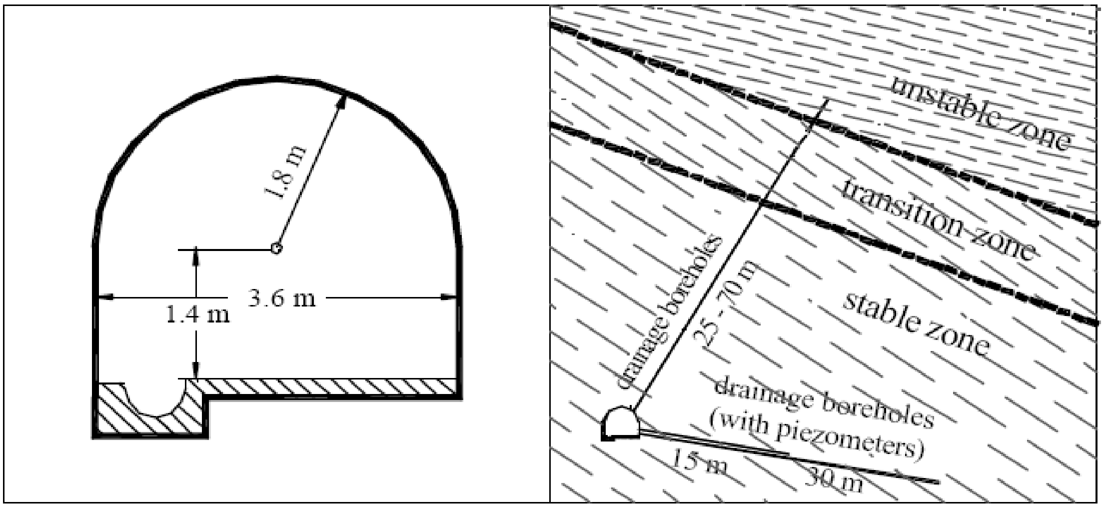
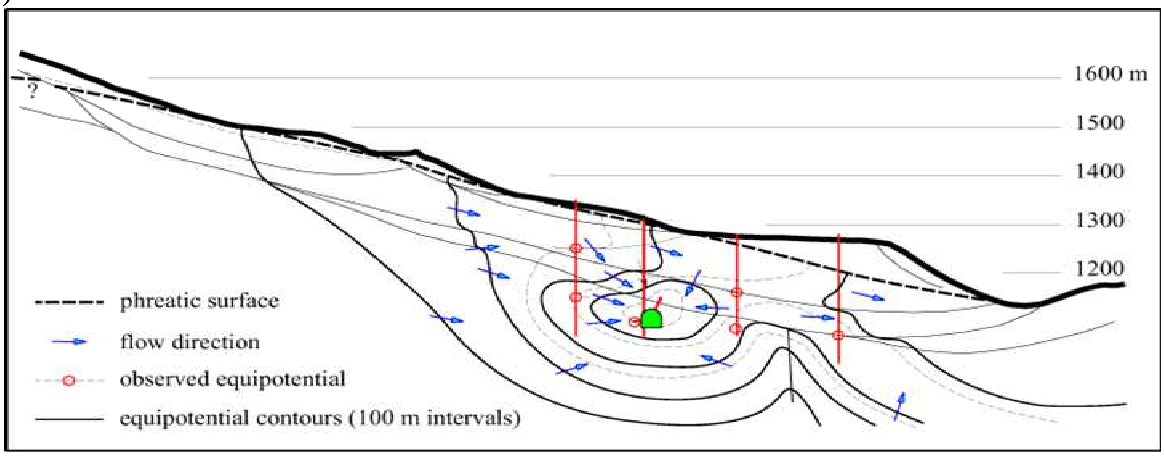


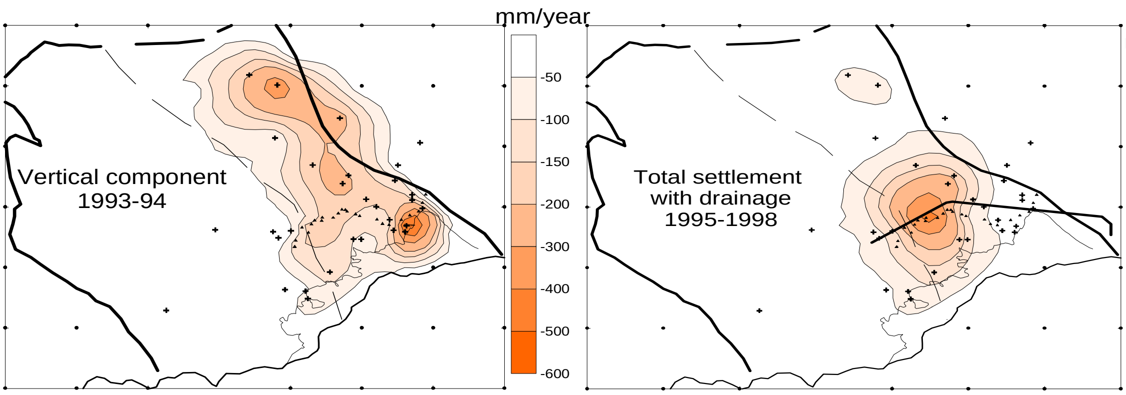
Figure C.3.3 - Plan view and block model of Campo Vallemaggia slide, with scale (after Bonzanigo et al., 2001).



(a)



(b)



(c)

Figure C.3.4 - Drainage gallery adopted for Campo Vallemaggia slide. (a) schematic representation; (b) 2-D hydrodynamic flow model of the lower Campo block due to drainage adit; (c) settlement measure before and after drainage (after Eberhardt et al. 2007, adapted by Springman et al. 2011).

C.3.3 PONTRESINA CHECK DAM

Pontresina is a municipality in the Oberengadin sub-district of Maloja in the canton of Graubünden in Switzerland. Pontresina has an area of 118.2 km^2. Of this area, 16.7% is used for agricultural purposes, while 8.8% is forested. Of the rest of the land, 1.6% is settled (buildings or roads) and the remainder (72.9%) is non-productive (rivers, glaciers or mountains). It is located in Val Bernina, which is the highest altitude valley that branches off the Upper Engadine Valley. It consists of the old village sections of Laret, San Spiert as well as Giarsun and the new sections on the mountain slopes (including Muragl). Nearby glaciers include the Morteratsch Glacier and the Roseg Glacier (www.pontresina.ch).

A debris flow and avalanche channel previously split the village in two parts, but mitigation works have diverted the channel around the village, allowing construction to fill this gap. Pontresina has escaped large-scale natural catastrophes in living memory. Far-sighted investment in hazard zone planning and avalanche and landslide shoring have made crucial contributions, but have required continual intensive analysis of natural hazards on the part of authorities.

The possible consequences of climate change were considered at an early stage and, Pontresina can now be regarded as a pioneering municipality with respect to permafrost, landslide and avalanche protection.

The Giandains Protection Dam, completed in 2003, protects the village of Pontresina from avalanches and the possible consequences of thawing permafrost (**Figure C.3.5**). The total area of the construction is 6.3 ha. and over three hectares of waste wood and young forest had to be cleared for the purpose. Permafrost is a widespread phenomenon above 2,500 m a.s.l. in the Alps. Combined with global warming, thawing permafrost can lead to various forms of mass movement from rockfalls and landslides through to debris flows. Whereas thawed rubble in loose debris increases volumes of debris flow, greater rock fall is to be anticipated in rock permafrost, causing landslides at unstable points (Arenson et al., 2002). Arnold et al. (2005) identified possible unstable situations during thawing of massive ice in permafrost that could lead to instabilities on slopes steeper than an interface angle of friction between ice and rocky debris cover in an active layer.

(a) (b)

Figure C.3.5 : Pontresina dam (www.pontresina.ch).

C.3.3 ARSCHELLA OST CREEPING LANDSLIDE

The mitigation measures adopted for this landslide are based on the observational method (Peck, 1969; Vollenweider, 2003). This slope had been creeping at the rate of 0.3 m/a, and had been identified as unacceptably unstable. Vollenwieder proposed that the equation in **Figure C.3.6** could be used to design measures to increase a global safety of factor by $\Delta\gamma$, dependent upon initial velocity v_0, "reduced design landslide velocity" v, an empirical factor ρ (obtained from temporal measurements), ρ_0=0.05 in this case.

A schematic representation of the force acting in a slope reinforced by anchors is reported in **Figure C.3.6**. It is possible either to reduce the loading by ΔS, or increase the resistance by ΔR to raise the factor of safety by $\Delta\gamma$. The mitigation measures adopted in this case included a double row of anchors. The anchor acts to increase the normal force acting on the soil and hence increase the mobilised resistance too. $\Delta\gamma/\gamma_0$=7-10% due to the anchor mitigation measures. The landslide velocity before anchor installation was 25-40 mm/a reduced to 0.3-1.3 mm/a after installation of the anchors.

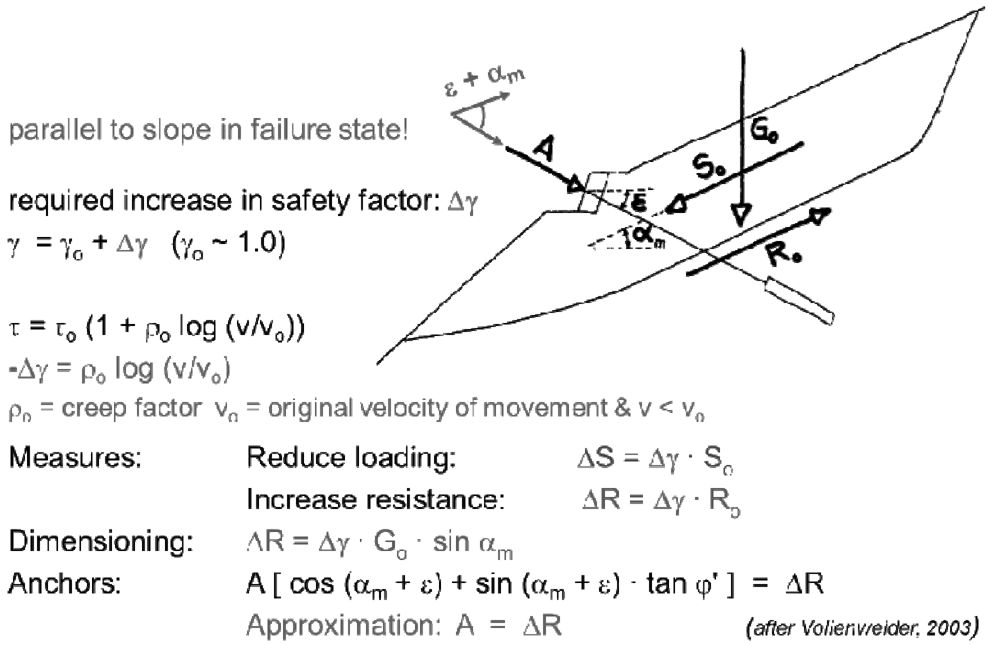


Figure C.3.6: Anchor mitigation measure: forces acting (after Vollenweider 2003).

Figure C.3.7: Arschella Ost Sedrun landslide (Springman et al. 2011).

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ANNEX D

INSURANCE POLICIES AND NATURAL HAZARDS IN SWITZERLAND AND EUROPE

D.1 INTRODUCTION

Natural hazards have caused considerable damage to humankind in Europe in recent years. Beyond social damages, the financial impact of structural damages to buildings and infrastructure is considerable (**Figure D.1**); for instance in Switzerland, the economic cost due to natural hazards in 2007 was estimated at 128 million Swiss francs and 417 million Swiss francs in 2008, only for buildings, according to Swiss Public Fire Insurances (SPFI), 2007 annual report. In financial terms, the insurance companies are, together with governments, one of the most important actors involved and are therefore particularly interested in reducing the financial impact of such disasters. The financial weight of the insurance industry gives it a strong influence in the field of risk mitigation.

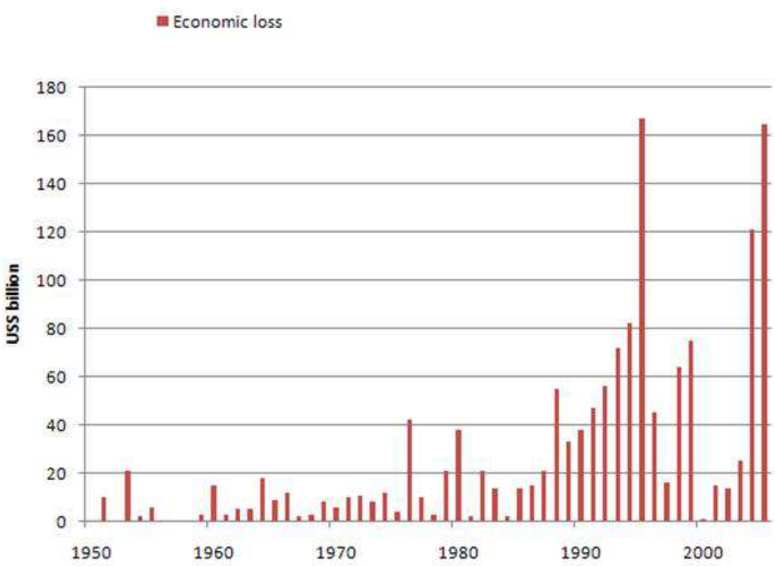

Figure D.1 Yearly cost for the main natural catastrophes in the world during 1950-2001 (vertical bars), including a trend curve. (Modified after GeoRisks Research Dept., Munichre, 2005)

This Annex is structured as follows: (a) the first part attempts to answer the question “why do we need natural hazard insurances and how are they involved in risk mitigation?” (b) the second part discusses the different possibilities for the insurance industry to reduce the financial impact of natural disasters and the potential that insurance companies have to anticipate them; This part will be followed by (c) an overview of the Swiss natural hazard insurance system, illustrated by three case studies; Afterwards, (d) will focus on the particular case of reinsurance and the role reinsurance companies can play in anticipating natural disasters. Finally, (e) we will present an overview of several systems of natural hazard insurance from different countries, including the differences between public and private insurance systems.

The increase of insurance costs over recent years is not only caused by the increased intensity of hazard, but also the increase of population density in urban areas and the increase of buildings vulnerability, their increased costs and the value of the property market (AEAI 2008). We will see later that insurances policies can affect each of these factors.

D.2 WHY DO WE NEED NATURAL HAZARD INSURANCES? THE PLACE OF INSURANCE IN RISK MANAGEMENT

D.2.1 CHARACTERISTICS OF NATURAL HAZARD INSURANCES

As noted by Smith and Petley (2009), the need for insurance arises when a risk is perceived and recurrent. The owner pays a fee (premium) that transfers the financial risk to a partner (insurer). If the premium is fixed at an appropriate rate, it will cover the eventual damage costs caused by an event. This action allows the policyholder to have guarantees to enable recovery of his goods after an event. However, commercial natural hazard insurances concern principally developed countries (**Table D.1**). *"80 per cent of all premiums for private insurance worldwide are paid in Europe or America"*. *"At present, there is a limited market for disaster insurance in the developing nations"* (Smith and Petley 2009). For example, only 2 % of the losses due to Hurricane Mitch which affected Central America in 1998 were covered by insurance. Despite this there are insurances for all kinds of disasters, but the existence of insurance depends on the number of insured concerned, it is necessary to have enough policyholders to be cost-effective.

Table D.1: Catastrophes in the world. A huge difference can be observed between events in Europe or in the rest of the world (Source Swissre 2006)

Insured Loss (in USD m)	Victims	Date	Event	Country
6802	110	25.12.1999	Winter Storm Lothar	Switzerland, UK, France et al
5157	22	15.10.1987	Storms and floods in Europe	France, UK, Netherlands et al
2621	38	06.08.2002	Severe floods	UK, Spain, Germany, Austria et al
3	138000	29.04.1991	Tropical cyclone Gorky	Bangladesh
258	10000	12.12.1999	Floods, landslide	Venezuela, Colombia
599	9000	22.10.1998	Hurricane Mitch	Honduras, Nicaragua et al

Natural hazard insurances have some particularities that distinguish them from other types of insurance (car, life, fire …). Specifically, the *occurrence frequency*, the *event size* and the *location*, are specific parameters of natural hazard insurance (Zimmerli 2003). Some comparisons with *fire insurances* can be presented to illustrate these specificities (**Table D.2**).

Table D.2: Summary of natural disaster insurance specificities for Fire and Natural hazards (Modified after Zimmerli 2003)

DIFFERENCE	**FIRE**	**NATURAL HAZARDS**
Occurrence frequency	High	Low
Event size	Individual risk affected (individual building or complex of buildings)	Large part of portfolio affectd (entire districts)
Location	Low importance	High importance
CONSEQUENCES		
Pricing	Minor fluctuations in the loss burden; therefore, burning cost analysis and exposure rating are sufficient	Major fluctuations in the loss burden; therefore, scientific models are required
Loss potential from single event	Low to medium	Very high
Geographical distribution	Minimal impact on losses, no accumulation control required	Major impact on losses, accumulation control important

D.2.1.1 Occurrance frequency

In case of fire insurance, the probability that a fire affects a single building is very low. On the other side, at the portfolio level, the chances that an event happens are important and rather stable over a given time period. This is quite different in terms of natural hazard insurance. Indeed, natural catastrophes are not frequent in a portfolio and can vary considerably over time. Then the need for anticipation and evaluation of future claims is strong for insurance companies. Nevertheless a catastrophic loss, threatening the stability of insurance, due to a major disaster is difficult to predict because major disasters are by definition at a larger scale than those which occurred in previous years. In this case,it is necessary to take into account a longer statistical period to evaluate the occurrence period. This statement is confirmed by Kuzak et al. (2004):

> *"The severity of these events is high because they are large-scale earthquakes or meteorological phenomena affecting thousands of square kilometres, sometimes impacting hundreds of thousands of properties, and since the frequency of these events is low, historical data is usually insufficient to estimate future monetary losses. In such cases, risk assessment needs to be prospective, anticipating scientifically credible events that could happen in the future, but have not yet taken place." (Kuzak et al. 2004)*

D.2.1.2 Event size

In most cases, a fire is a very localized event, while most natural catastrophic events affect a larger part of a portfolio, and not only a single object of the portfolio. In the case of floods and landslides, an entire district may be affected. With regard to fire insurance, the prescriptions and regulations on protection against fire may help to confine the fire and thus avoid it spreading to a disproportionate extent. Gurenko (2004a) wrote:

> *« Even in industrial nations with well-developed insurance markets the loss potential from catastrophe risk exposures can be so large that the insurance markets are unable to provide sufficient capacity at acceptable price. » (Gurenko 2004a)*

This is confirmed by Swissre:

> *« The sum of all claims can reach considerable amounts and far exceed the amount of premiums collected during one year. » (Zimmerli 2003)*

On the other hand, this is what the insurance companies claim because they have to justify premium.

D.2.1.3 Location

The spatiality parameter has an influence on the vulnerability of a portfolio. Indeed, with appropriate modeling, natural hazards can be localized in space, creating hazard maps. This is not the case for fire which can strike at any place within an insured portfolio. One way to do is to adapt the buildings to identified natural hazard in order to reduce vulnerability.

As aconsequence, it is essential for an insurer to be sure that the type of properties insured are varied and that the geographical distribution is spread. In this way, only a part of the portfolio is concerned by a specific disaster and only a fraction of the portfolio can be destroyed by a single event (Smith and Petley 2009).

D.2.2 ROLES OF THE NATURAL HAZARD INSURANCE IN THE CONTEXT OF RISK MITIGATION. PREVENTION VS REIMBURSEMENT

Kelman (2003) provides a simple and understandable definition of natural hazard insurance:

> *"Insurance involves many people with each individual paying a small amount of money, yielding a large pot of money. When a disaster affects a small number of people, the pot is available to give large sums to small affected population." (Kelman 2003).*

Insurance intervenes at the moment of financial compensation for damages and allows victims to rebuild after a disaster. Thus, insurance provides cash to allow rehabilitation. This can significantly improve the recovery phase of disasters at a time of extreme stress and thereby reduces disruption of normal life (Walker 2005). This financial compensation acts on the resilience of a devastated society, but depends on the financial capabilities of insurance companies to cope with disasters. However, insurance companies have also a role to play before the event, by financing preventive measures (**Figure D.2**).

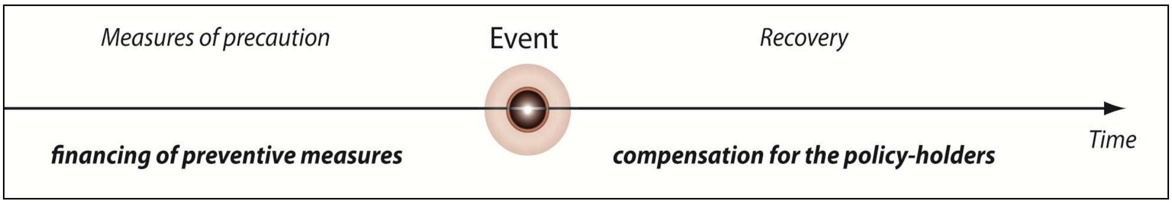

Figure D.2 The insurance operates on two levels, before and after the event: financing of preventive measures and compensation for the policy-holders.

Worldwide, insurance companies are facing rising costs due to natural disasters (Munichre 2002). To act on the financial impacts of disasters, insurance companies can proceed in different ways. Damage assessment by modeling the different components leading to financial compensation of victims is one of the possibilities to act on rising costs, it is in all cases the first necessary step to a better understanding of risk. According to Khater and Kuzak (2002), these components can be described by three different modules, regardless of the kind of natural hazard: the *hazard*, the *damages* and the *loss* (**Figure D.3**). These three parameters are described in the following points.

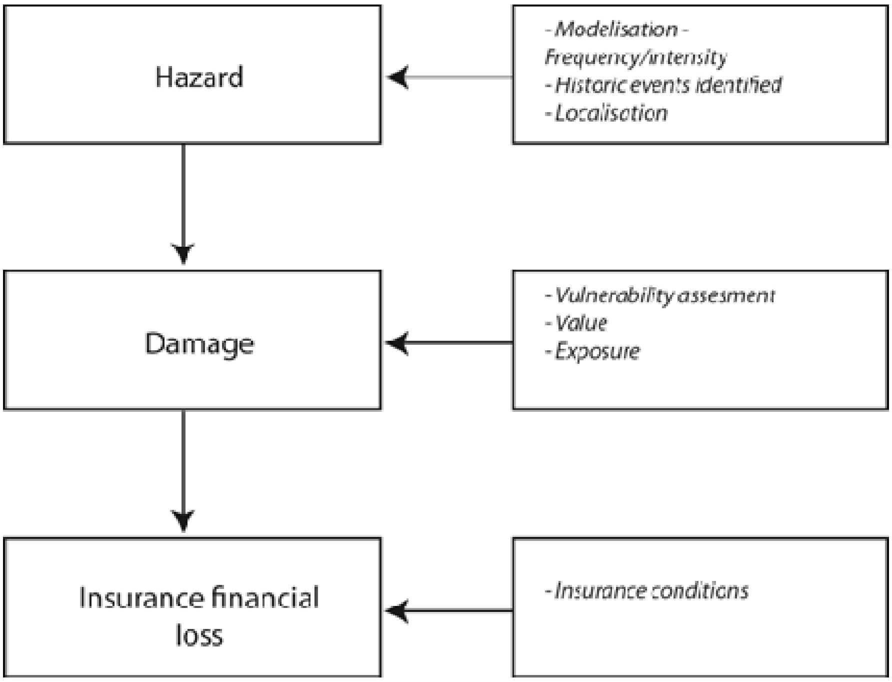

Figure D.3 Component of a risk model. Modified after Khater and Kuzak (2002).

D.2.2.1 Hazard module

With its financial weight, the insurance industry can finance mitigation measures. Furthermore insurance companies can participate in research about hazard assessment, the first necessary step before any mitigation. Although some insurance companies or reinsurance companies have their own research laboratory or provide funding to research in the field of natural hazard modeling and understanding (see for example Willis research network 2010).

Insurance companies also have the possibility to reduce risk by financing protective measures. This last possibility may have the form of a public-private partnership in the case of private insurance (Gurenko 2004b). In fact, the insurance companies, like the other risk partners (policyholders, reinsurers or government), must take the responsibility for a portion of the risk and some of the costs (Munichre 1997).

Whatever the method used to protect properties exposed to natural hazards, a residual risk remains. This affirmation is demonstrated by the analysis of past events (for example BAFU 2007) where the protection measures were exceeded. This residual risk is on one hand linked to the possibility that protection measures may fail or may not work as intended. On the other hand the residual risk is linked to the possibility that the event exceeds the chosen level of protection. Many European countries, governments and insurance companies are now thinking in terms of vulnerability reduction by decreasing residual risk, since this reduction can have major consequences in financial terms.

D.2.2.2 Damage module

As already shown in Figure D.1, the cost associated with natural damages has increased during the last decades worldwide. However, this does not imply that the number of events has increased everywhere. For example concerning buildings, even if the damage costs have increased since the 1990's in Switzerland (AEAI 2008), the number of events is relatively stable (AEAI 2008).

> *« This increase (in economical cost) is principally a result of higher population densities, a rise in insurance density in high-risk areas and the high vulnerability of some modern materials and technologies. » (Zimmerli 2003)*

Both an increasing population and an increasing number and costs of infrastructure contribute to rises in costs. The following reasons may also explain the increasing costs of damage in recent years: (1) negligence in the consideration of danger zones (authorities and project managers) and land use planning, (2) extension of building zones in risk areas, (3) soil waterproofing or (4) increasing of the buildings value. All these factors have contributed to raise the costs of damages.

Insurance companies have many possibilities to approach the problem. First, they can act directly on the financial statement by increasing premiums or by decreasing allowances (**Figure D.4**). This solution affects the services provided to the property owners without directly decreasing the potential damages.

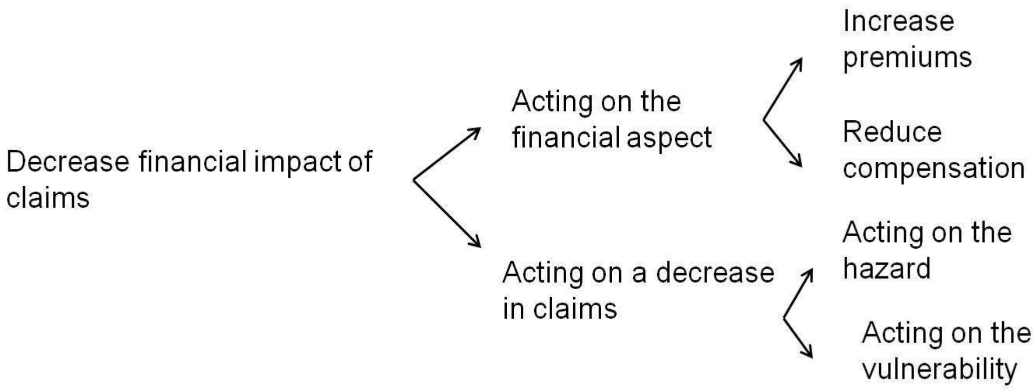

Figure D.4 Opportunities for insurance companies to act on costs from natural hazards

Another possibility for insurance institutions is to act on the number of claims and/or their importance, trying to reduce the causes of the disasters; adapting buildings and thus influencing the vulnerability. This aspect is related to the loss module, presented in the following point. Taking into consideration the vulnerability of an insurance system has many advantages:

- Insurance can better estimate the annual cost of damages by assessing the vulnerability of its insured property (Kelman 2003).
- The owner is aware of the vulnerability of his property and will seek to reduce it, often by simple measures. Being aware of the fragility of his property, the owner will be better able to respond to an event.
- Simple measures to reduce vulnerability can be undertaken only from the time that the fragility of the object was evaluated. From this moment, this is possible to consider the best solution from a cost-effectiveness point of view
- With a system of encouragement by the insurance companies, the owner could be motivated to undertake preventive measures. That can significantly reduce the amount of damages.

Kelman (2003) proposes an insurance system oriented towards vulnerability mitigation, called « Reverse insurance ». This system is based on an incentive to reduce vulnerability and differs radically from the systems used in major European countries. It is not the owner who pays to be insured, but the insurance (or government) who provides assistance to the insured to reduce the vulnerability of its property. It is therefore an inverse insurance system where the owner receives funding to reduce its vulnerability, while the amount of post-disaster compensation is reduced:

> *"Each year, the government could pay each individual a small amount of money which should be used by each individual to reduce their own, and their community's, vulnerability. In exchange, post-disaster assistance from the government would not be as extensive as before. The government, though, must provide information, advice, and assistance on techniques for individual and community vulnerability reduction. This system operates to some extent when governments provide grants or funding for disaster mitigation activities,*

> *but cases are rare where every citizen or every community receives funding and is responsible for their own vulnerability reduction.”* (*Kelman 2003*)

Kelman then shows the following benefits to its proposal:

- *“The government can better estimate the cost of disasters to government each year.*
- *The system encourages locally-based vulnerability reduction and encourages efficient innovation. Individuals and communities are given the resources to decide for themselves the vulnerability pathway to choose. They suffer the consequences of their own decisions rather than suffering the consequences from someone else’s decisions.”* (*Kelman 2003*)

Some limitations and defaults are pointed out by Kelman (2003):

- *” For each individual, if the disaster happens in 50 years, they would have had time to reduce their vulnerability. If the disaster happens in 1 year, they would be in trouble.*
- *A challenge exists in ensuring that accurate, understandable, and effective “information, advice, and assistance on techniques for individual and community vulnerability reduction” are provided by the government, particularly given the diversity of groups and vulnerabilities which always occur.*
- *A challenge exists in ensuring that people do use the payments for vulnerability reduction. If neighbours or neighbouring communities choose different uses for the money given to them for vulnerability reduction, problems for everyone may result.*
- *Reducing post-disaster assistance could cost lives and may be politically dangerous.*
- *The payments would be calculated partly based on the magnitude of the event expected. If that event magnitude is exceeded, then the government would be obliged to assist fully. ”*

D.2.2.3 Loss module

Financial insurance loss is determined by insurance conditions (Khater and Kuzak 2002), which include the *deductibles* (portion of the claim that is not covered by insurance), the *limits* (maximum value of loss take into account by the insurer) and *the total insured value* (effective value engaging the insurer) (**Figure D.5**). The insurance conditions determine the consideration of financial damage by the insurance, if it is complete or partial.

By influencing insurance conditions, insurance companies can act directly on the financial statement by increasing premiums or by decreasing allowances.

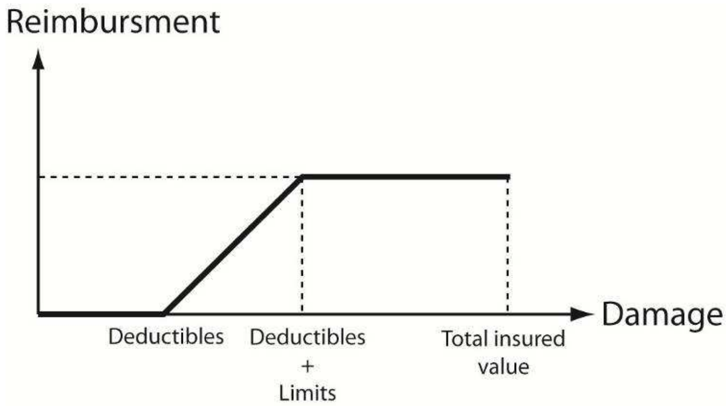

**Figure D.5: Relation between damage and reimbursement.
Modified after Khater and Kuzak (2002).**

Modeling the loss is difficult, because it has to take into account the evolution of vulnerability, land use planning, environmental conditions and the increase of population.

> *« Therefore, what is needed is a model that is prospective in risk estimation, not retrospective." (Khater and Kuzak 2002)*

Besides, the insurance company can become more cost-effective by a variety of other financial measures (Smith and Petley 2009):

- Re-rating premiums
- Restricting cover
- Widening the policyholders base
- Transfer the risk to a reinsurance

D.2.3 WHICH GOVERNANCE FOR INSURANCE COMPANIES?

As we have seen, natural hazard insurances participate in the financial recovery after an event. This intervention is necessary, because in the case of natural hazard event, the damages can be so important that it can be impossible to recover the same state as before the event without financial help. Insurance companies can thus play the role of the State without altering the economy of the country. Therefore, an insurance system is a necessity to protect the local economy. As shown later on (chapter D.5), insurance can be public or commercial, both systems have advantages and drawbacks.

From there, we can wonder which governance the insurance industry can promote. In the case of the natural hazards, it can intervene in several ways on the function of the state. Besides, a lack of insurance can discourage development in hazardous areas (Smith and Petley 2009).

The insurance sector represents an important lobby that can intervene politically. This lobby has the possibility of proposing regulations and it can take part in land use planning. This will depend of course on its will to get involved in the prevention.

The insurance sector has an important financial weight that gives power but also responsibilities. An insurance company is often a giant in the local economy. However, a menace threatens its position of strength: self-sufficiency. This occurs when the insurance company pays without necessarily seeking to reduce the amount of damages, as long as its financial mass is sufficient. As such, insurance is not an incentive system to reduce disaster costs, because after every disaster, the owner is reimbursed. The owner has therefore no incentive to reduce its vulnerability. In other words, if the property is damaged, the insurance will restore it in the initial conditions. By consequences this insurance is condemned to assume the increasing costs caused by the increase in claims.

By requiring obvious and defined protection goals, the insurance companies have the possibility to control the fragility of the portfolio. They may thus decide of the fragility degree of the portfolio and the "damage tolerance". They have the option to require a kind of label taking into account the exposure of the property, but also its vulnerability. This option might help stabilize the rising cost of damages.

D.3 THE ROLE OF INSURANCE IN SWITZERLAND AND ITS INTERVENTION IN RISK MANAGEMENT. CASE STUDIES.

D.3.1 SWISS INSURANCE ORGANIZATION

As Smith and Petley (2009) noted, risk management is the process through which different strategies are evaluated in order to mitigate threat and to manage economic losses. Traditionally, the national government leads this management. As seen before, insurance companies have a role to play in risk management. With a strong presence in Switzerland (100% of buildings covered by insurance in certain regions), they are involved in the risk management.

As a federal political system, Switzerland does not have a unique insurance system (**Figure D.6**). Indeed, each canton has a different insurance policy against natural hazards. 19 of the 26 cantons have a system of cantonal monopoly public insurance for buildings. The Cantonal Insurance Institutions (CII) are independent of political power but are obligatory for owners who must ensure any building. The CII have an inter-cantonal reinsurance pool (UIR), which works like a reinsurance company, but specifically for the CII. The CII is involved in the allocation of building permits in risk areas. Through the CII Association (AEAI), it makes recommendations on the consideration of natural hazards in constructions (Egli 2005 and 2007). AEAI also finances a foundation of natural damage prevention, which funds projects in the field of risk mitigation. For instance the current project carried out by University of Lausanne and the societies R&D, Bianchi Conseils and Risk and Safety "*Analysis Tool to assess buildings vulnerability to flooding and risk reduction*" aims to provide an accurate assessment of building vulnerability to flooding and to propose mitigation solutions (Choffet et al. 2009). The AEAI also provides educational courses for construction professionals.

In addition to CII, private insurance companies cover cantons which do not have public insurance, together with most of the furniture and goods not insured by the CII. Thus, private companies are also active in the field of natural hazard management to reduce the costs of damages. In addition, they participate in financing mitigation measures, as this will be shown in the example below (section D.3.2). The cover extent and the amount of the premiums in Switzerland are uniform and compulsory for all the private insurers. Considering its great sociopolitical and economic importance, this principle was registered in the law in 1993 (OFAP 2008).

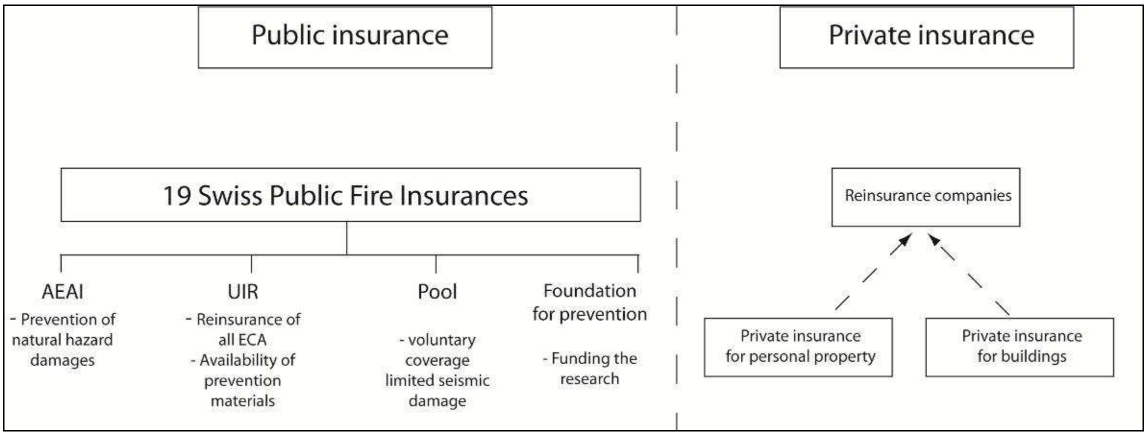

Figure D.6 Swiss insurance system for fire and natural hazards.

A few years ago, the main activity of CII was fire insurance. Today, these institutions annually compensate 18'000 fire claims for a total estimated at 270 million Swiss francs (AEAI 2008). Thanks to an effective strategy for prevention and intervention (Figure D.7), the trend in the number of claims is stable or has even declined in recent years. Nevertheless, since about 2004, the trend of the economic losses due to natural hazards has become more important than the same trend due to fire. The prevention policy against fires conducted by the CII has proven to be effective and there has been a decrease in the annual financial amount of damages (**Figure D.7**), but the same principle is not observed for natural hazards, where the economic losses caused by natural elements have grown. This fact illustrates that, on one hand, the reduction of fire risk was achieved through vulnerability reduction by adapting buildings to new standards. On the other hand, for natural hazards, the prevention has focused on reducing hazard, which has given fewer results. This fact shows that measures focusing on vulnerability reduction can be efficient.

Damages due to natural disasters are very expensive for private insurers, since insurance companies are so active in financing protection measures in threatened areas. The Swiss insurance company *La Mobilière*, responding to the exceptional floods of 2005, has decided to create a fund of ten million Swiss francs to fund mitigation projects. This amount has been doubled since then. More surprisingly, this company also decided to participate in funding a research center on global warming effects in alpine areas at the University of Bern for five million francs. Even if these measures are also part of a communication and advertisement strategy, they allow nevertheless some progress to be made in research.

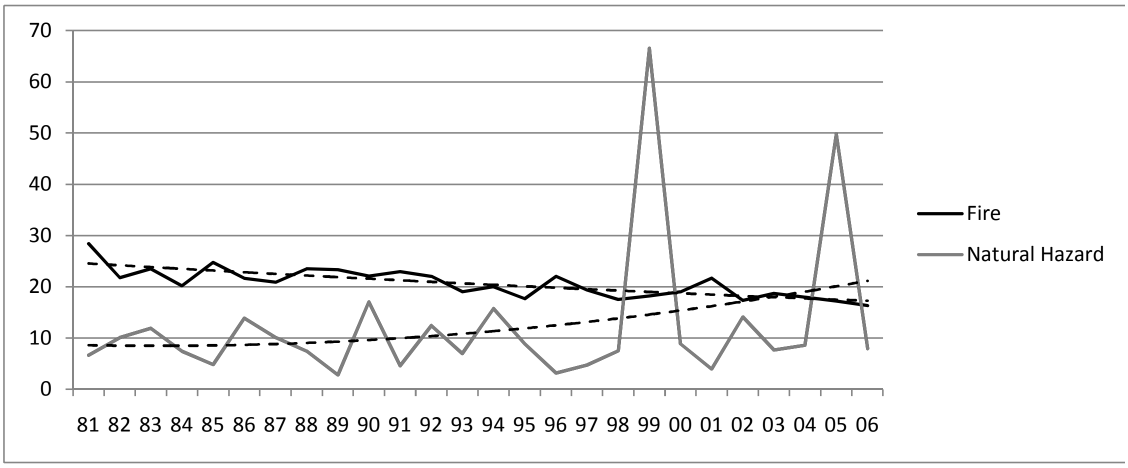

Figure D.7 Evolution of fire damages and natural elements of the 19 CII. The Y axis represents the centimes by 1'000 francs assured, the X axis represent the years. The dashed lines illustrates the trend. Modified after AEAI (2008).

D.3.2 CASE 1: THE PRIVATE INSURANCE SYSTEM IN VALAIS

The canton of Valais (**Figure D.8**), located in the Swiss Alps, has a strong presence of natural hazards in its territory. Each year avalanches, debris flows, rock falls and floods do extensive damage to infrastructure.

The canton of Valais does not have any system of public insurance (**Figure D.8**). Only private companies insure goods and there is no obligation for an owner to be insured. A first consequence is that the insurance penetration rate is low, meaning that only a few people ensure their property. Since the number of policyholders contributing to the common pot is reduced, the insurance cost is more expensive.

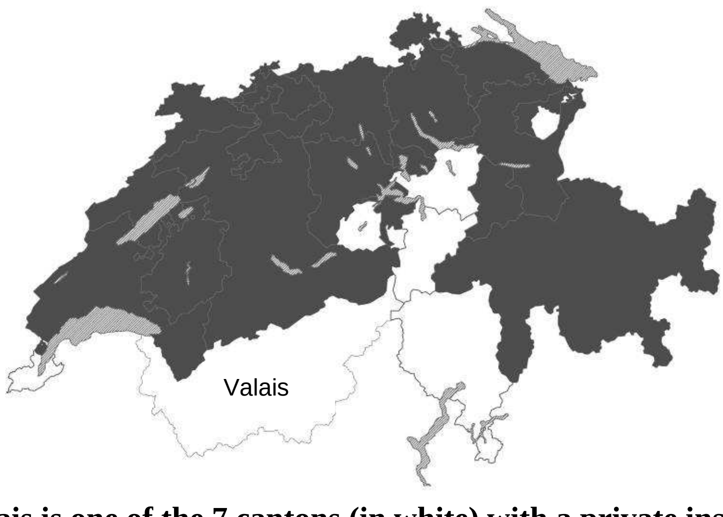

Figure D.8 Valais is one of the 7 cantons (in white) with a private insurance system. The other cantons have a public insurance system.

Figure D.9 The canton of Valais is threatened by several natural hazards. For instance, the catastrophe of Gondo in October 2000 killed 13 people. The economic loss for the whole canton was estimated in more than 670 million Swiss francs. Image source: www.vs.ch.

Another consequence is that private insurance companies are more present in this region and play a major financial role. A case illustrates the implication of private insurance industry in the risk mitigation: Port-Valais (www.mobi.ch). The Municipality of Port-Valais, located in the Rhone Valley, is threatened by mudflows and landslides (**Figure D.**89). The insurance company *La Mobilière* financed part of the construction of two dams. These dams are intended to protect residential areas and industrial and artisanal settlements. Thanks to this financing, the insurance company expects savings on possible damage costs in its portfolio. It estimates indeed that the possible costs induced by a landslide would be higher than the insured value present in the landslide area and that the protection measures are cost-effective. In the contrary case, an individual reduction of the risk would have been privileged, by the buildings adaptation or by financing relocation, as that can be the case elsewhere in the canton. The last solution would have perhaps been that the insurance company would have quite simply chosen not to assure the values concerned. In fact, private insurance companies have no obligation to insure everything and that is the main difference between private and public sector in the field of insurance.

Concerning the particularity of private natural hazard insurance, private insurance for natural damages is regulated by a federal office (Federal office of the private insurances). Thus, such as CII, the proposed insurance is offered at a uniform rate in exposed areas (OFAP 2008).

D.3.3 CASE 2: THE PUBLIC INSURANCE SYSTEM IN NIDWALDEN

Since the canton of Nidwalden is frequently affected by different kinds of natural hazards (see for instance **Figure D.10**), its population is very concerned to these phenomena. Regarding buildings, the floods of 2005 have cost 120 million Swiss francs (BAFU 2007). Since the population of this canton is reduced (40'200 people, 276.1 Km^2), the assured community is small, meaning that the damages have strong financial impacts. The Swiss public insurance system allows remedy of this problem through a system of solidarity between CII. Thus, a policyholder of another canton pays a small amount for a policyholder of Nidwalden touched by a catastrophe.

Figure D.10 Nidwalden is affected by natural hazards. Here, a shallow landslide at Ennetbürgen during the events of 2005. Image source: Nidwalder Sachversicherung.

To reduce the financial consequences of natural hazards, CII Nidwalden has an advanced prevention policy, oriented on vulnerability reduction.

For example, the CII is working with the authorities to provide recommendations in buildings construction. It edits technical recommendations for owners in hazardous areas (**Figure D.11**). It has also employed full-time workers in the field of natural hazards prevention for many years. They have developed several strategies to assess the risk. Nidwalden is also far ahead of other cantons in detailed natural hazard mapping. Indeed, it does not only map the extent of the phenomena, but also the intensity, the risk and all event information is collected in databases.

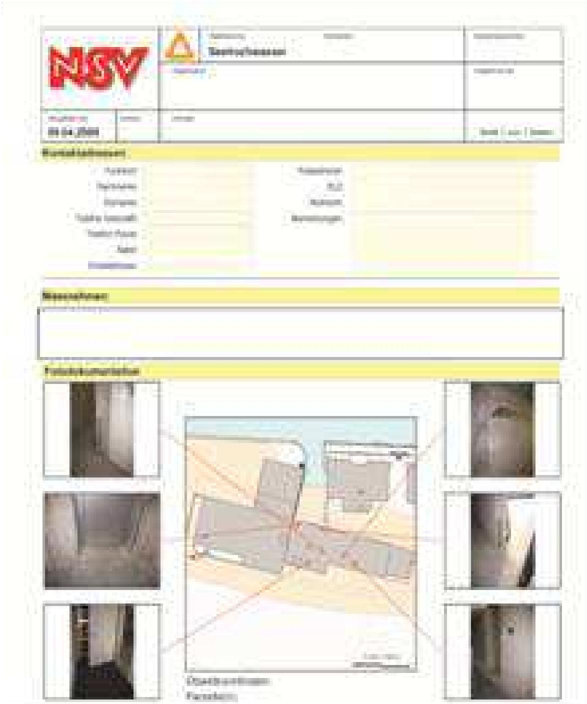

Figure D.11 The Nidwalden CII offers remediation solutions through technical sheets. Image source: Nidwalder Sachversicherung.

D.3.4 CASE 3: EXAMPLE OF A FAILURE. THE CASE OF THE FALLI-HÖLLI LANDSLIDE NEAR FRIBOURG

CII of Fribourg participates in delivering building permits. Having an obligation to insure all buildings on the cantonal territory, it gives an expert advice on building implementation. However, a noticeable example in the early 90s illustrates the limits of the influence of insurance on the amount of damages due to natural phenomena.

In May 1994, a major landslide of 40 million m^3 was activated in the Prealps zone, at the location of Falli-Hölli. The dimension of the unstable mass was 2 km long and 700 m wide. The landslide covered an area of 1.5 km^2 and an estimated depth of 60 m. The maximum displacement rate was measured in early August at 6m/day (Caron et al. 1996, Raetzo and Lateltin 1996).

Before the landslide occurrence, the CII Fribourg showed its opposition to the construction of a touristic area in this region already recognized as unstable. However, underpolitical and economic pressure, the authorities of this time didn't take into account the negative notice of the CII Fribourg and authorized the construction of this holiday village. The landslide destroyed 41 houses causing economic losses estimated at 15 million Swiss francs. Despite its negative notice, the CII of Fribourg was required to pay for the financial damages. This illustrates that the insurance companies have no political role and their decisions can be contested by political decision maker. In fact, CII have the obligation to insure all buildings, despite its own notification.

The case of Falli-Hölli illustrates the limits of the possible involvement of insurance in prevention (**Figure D.12**). This event triggered the writing of the prevention policy in Switzerland by the federal recommendations on natural hazards.

Figure D.12 The landslide of Falli-Hölli destroyed 41 houses causing economical losses estimated in 15 million Swiss francs. Image source : Hugo Raetzo.

D.4 REINSURANCE COMPANIES AND NATURAL HAZARDS

D.4.1 THE ROLE OF REINSURANCE COMPANIES IN RESEARCH

An insurance company can transfer, against payment, part of the risk of a premium to a reinsurance company. A reinsurance company is somehow the insurance of the insurance companies. It will directly cover the damages exceeding the insurance provisions. The reinsurance companies are thus very interested to estimate the potential damages induced by natural disasters. These companies are very active in the publication of prediction of risk and natural disasters. Contrary to private insurances active at the national level, the companies of reinsurance work on the worldwide market and are consequently interested in catastrophes in a more global manner. Swissre and Munichre, the leaders on the reinsurance market have their own publication services regarding natural hazard prediction and scientific model development. The reinsurance companies finance scientific studies (Bock and Seitz 2002) or finance research work, for example the UIR (reinsurance of swiss public insurance) in Switzerland which publishes post-event reports (for example Imhof and Heuberger, 2008). They also finance scientific organizations, such as the Willis Research Network, supported by Munichre and Swissre (Willis Research Network 2010). This partnership between academia, public policy institutions and the insurance industry has the objective to lead scientific understanding of extreme events. For instance, this year Willis Research Network convened one of the largest sessions at the Annual General Assembly of the European Geosciences Union (Natural hazard Risk assessment Session, 55 papers from Insurers and Academics, Vienna, 2010).

D.4.2 FROM PREDICTION TO PREVENTION

Hurricane "Andrew" in 1992 and more recently hurricane "Katrina" in 2005 shook the reinsurance industry. Various insurance companies (including some important ones) had financial problems following these events, and the amount of damages were not anticipated by many insurers. Hurricane Andrew induced an insured loss of 16 billion dollars and is credited for the bankruptcy of ten insurance companies (White and Etkin 1997; Kunreuther 2001). The provisions and the covers of insurance were often not adequate. These two major catastrophes illustrate the need for the insurance and reinsurance companies to have better natural hazard models, in order to anticipate the most important catastrophic events and to estimate the maximum potential loss. This is more important for reinsurance companies, which must face mainly these kind of events. As seen up to now, the increased damage is not only due to the increasing hazard, but also influenced by other risk factors, such as the vulnerability and the increase of objects at risk located in hazard area. It is thus necessary to anticipate all the risk factors.

For the traditional insurance such as automobile, fire or life insurance, the first step of the potential loss model passes by a statistical study based on history:

> *“However, the risks of natural disasters are generally low-frequency, high-severity events.” (Khater and Kuzak 2002)*

Regarding natural hazards, it is not sufficient to anticipate the “normal” catastrophe, but it is necessary to anticipate “the worst” possible events. This is why reinsurances companies develop catastrophe risk models:

> *“Using current computer technology and the latest earth and meteorological science information, catastrophe risk models of earthquake or other perils (…) have been developed by specialist consulting companies. These models are now deemed essential tools for use by insurers, reinsurers and government agencies around the world to assess the risk of loss from such catastrophes.” (Khater and Kuzak 2002)*

The catastrophe risk model (**Figure D.13**) combines the components leading from the risk to the loss, described in the chapter D.2.2.

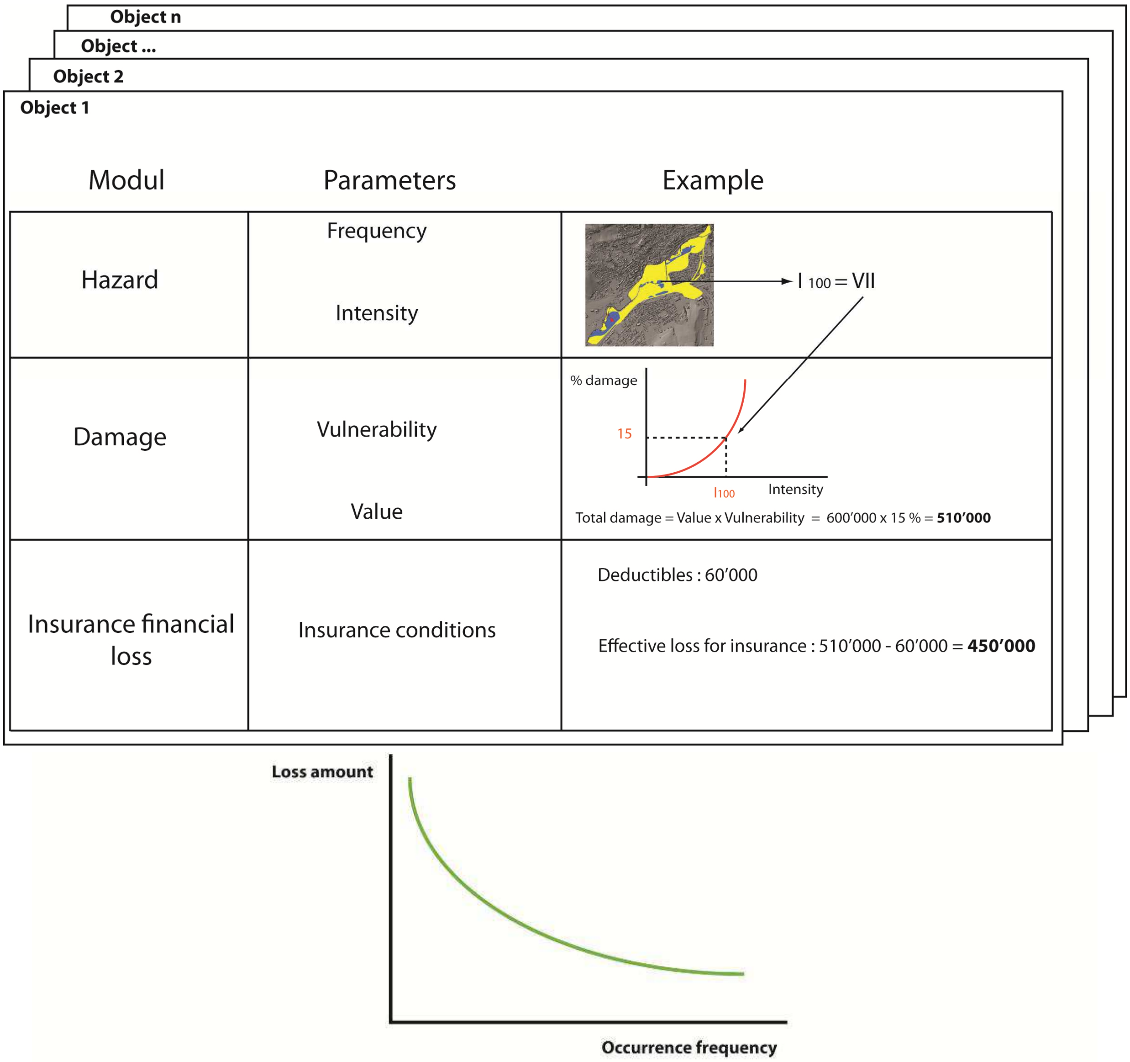

Figure D.13 Loss amount vs. frequency of occurrence. Summing the economic losses for all the objects gives a model of a catastrophic loss. Modified after Khater and Kuzak (2002) and Zimmerli (2003).

Once the risk model is established, it is possible to pass to a targeted prevention. International institutions, such as reinsurance companies, have shown their interest in promoting cost-effective mitigation measures to reduce the damage to property and infrastructure after a major catastrophe (Kunreuther 2001). Indeed, many possible benefits for insurance companies to encourage mitigation measures exist, as shown byn Kunreuther et al. (2004):

- (a) Reducing direct losses: Mitigation measures can avoid physical damages caused by the disaster to insured infrastructures as well as the loss of lives. For example for rock falls, building a reinforced wall can avoid building collapses and save lives.
- (b) Reducing indirect losses: This concerns the loss induced by the catastrophe but not directly to the infrastructure. This can be a long-term loss, for example a business interruption, causing a loss other than the direct loss.

- (c) Reducing losses to neighboring structures: A mitigation measure can avoid damage to other infrastructures, without having been designed for the neighborhood. For example, a building collapse can damage other buildings that would have been left standing otherwise. Mitigation measures that avoid the collapse reduce also the loss to neighboring structures.
- (d) Reducing financial costs from catastrophic losses: the mitigation measure can reduce the catastrophic losses and thus avoid the recourse to public finance envisaged in the case of great catastrophes exceeding the financial capacities of the private insurers.

D.5 OVERVIEW OF DIFFERENT NATURAL HAZARD INSURANCE SYSTEMS IN SEVERAL COUNTRIES

In August 2005, the whole northern Alpine region was affected by extreme floods. The total amount of damage to buildings, infrastructure and agriculture has been considerable. The insurance companies' data provide a first insight into the different insurance policies of European countries and how the proportion of insured loss can influence the overall loss (**Table D.3**).

For the 2005 events, Switzerland had the highest proportion of insured loss compared to the overall loss. Switzerland had the highest overall loss too. On the 2'100 billion USD of overall loss, 1'250 billion USD were insured (Munichre 2006). In Germany, the insurance cover policy is lower than in Switzerland and 50 billion USD of the 220 billion of overall damages were insured (Munichre 2006). In Austria, the proportion between overall loss and insured amount was even lower, because only 21% of the overall damages were insured.

Table D.3 Flood in the Alpine region in the summer 2005.

Country	**Overall Loss (US $ m)**	**Insured loss (US $ m)**	**%**
Switzerland	2'100 (Munich Re 2006)	1'250 (Munich Re 2006)	**59 %**
Germany	220 (Munich Re 2006)	50 (Munich Re 2006)	**22 %**
Austria	700 (Munich Re 2006)	150 (Munich Re 2006)	**21 %**
Europe	3000 (Swiss Re 2006)	1'700 (Swiss Re 2006)	**56 %**

Some countries (like Switzerland) have a strong insurance penetration in the society and so a high cover. Moreover, after a catastrophic event, an owner will not hesitate to declare as lost or damaged any object, event if it is only slightly damaged, considering that he is paying significant insurance premium. Both factors will increase the insured loss. In countries where insurance penetration and cover are lower, only completely destroyed objects will be declared damaged and the overall loss will be lower too

Since each country has a specific way to manage their reimbursement policy after a natural event, it is difficult to provide a general overview of all European systems. The insurance varies according to the risk or the insured property.

> *“The availability of landslide insurance is quite variable. In many countries, including the UK, private insurance for mass movement hazards is not available because of the risk of high numbers of claims” (Smith and Petley 2009).*

However, there exist two main categories: public insurance with a financing of the state and private insurance with commercial goals. In most cases, these systems are nested. Some examples with France, Turkey and Germany can be presented. The case of Switzerland was already described in chapter D.3.

D.5.1 THE CASE OF FRANCE

In France, natural hazard insurance is not obligatory. There is a government approach to insurance, which involves a mandatory extension to property insurance policies provided by private insurance companies. Insurance companies do not insure all natural risk and do not insure the values which could be concerned by a too large risk and which could affect the community pot in a too important way. The natural damages caused by a natural disaster are difficult to estimate. This is why the French state gives its guarantee, by the intermediate of public company, the CCR, which plays the role of a reinsurer.

France has a particular way to manage its prevention against natural hazards. Indeed, for the insured properties, a prevention fund exists (Fond Barnier) for the major natural risks, which finances measures of vulnerability reduction such as the expropriation of goods exposed to certain natural risks or to help the stricken regions. This fund is financed by a portion of insurance premiums. According to Mission Risques Naturelles (2004), this fund can be use for:

- "The acquisition by mutual agreement by the state, a municipality or a group of common properties strongly affected by natural disasters.
- The acquisition by mutual agreement by the State, a municipality or a group of common properties exposed to some major natural hazards which seriously threaten human lives.
- Measures to reduce vulnerability prescribed by a risk prevention plan (PPRN) to existing assets in risky areas.
- Studies and prevention work against natural hazards to the owner of regional authorities with a NRPP prescribed or approved "(Mission Natural Hazards 2004).

All compensation provided by insurance companies are subject to two prior conditions: (a) the French government must declare a state of natural disaster (being the government who decides whether or not an event is a natural disaster) and (b) the damaged property must be covered by a property insurance policy.

D.5.2 THE CASE OF TURKEY

Turkey Catastrophe Insurance Pool (TCIP) was established in 2000, after the earthquake in the Izmit area (1999). TCIP is a government-backed insurer. It has variable rates based on risk.

> *”The government of Turkey has decided to enforce the earthquake insurance on a nationwide basis with the sole purpose of privatizing the potential risk by offering insurance through the TCIP and then exporting the major part of this risk on the international reinsurance and capital markets. Initially funded by the WorldBank, TCIP was founded on 08.08.2000 and the program became effective since then. All registered residential dwellings that are located within municipality boundaries are required to be in the compulsory earthquake insurance coverage. With its 2.7 million policy count as of April 2008, Turkish Catastrophe Insurance Pool has a potential to become the largest earthquake insurance company in the world” (Yucemen 2008).*

So far, Turkey has a modest penetration rate of insurance. In Istanbul, the market penetration is approximately 27.3%” (Freeman 2004). TCIP covers only residential buildings. Freeman (2004) exposes some problems, which illustrate the weakness of this kind of insurance:

> *”From 2000 and 2003, fifty earthquakes occurred in Turkey and the TCIP paid total damages of 7 million dollars to 4200 homeowners. For the two most serious earthquake, the Government of Turkey waived the provisions of the Disaster Law requiring the purchase of insurance and declared all citizens eligible for government support, insured or not. The costs of non-insured victims in the 2002 and 2003 earthquakes cost the Treasury on additional 200 million dollars” (Freeman 2004).*

D.5.3 THE CASE OF GERMANY

Germany is a case of pure private insurance system with individual premium calculation. Since 1991, natural hazard insurance companies provide supplemental contract which covers economic losses due to floods, torrential rain, debris and mud flows, earthquakes, land subsidence, avalanches and snow buildup (Thieken 2006). However, apart from storms and hail, the insurance penetration rate against natural hazards is very low (95% vs. below 10%, respectively) (Schwarze 2010). This rate is very low in comparison with Switzerland or France, which have a governmental approach. This illustrates the fact that in case of private insurance system like in Germany, only those most threatened by natural disaster are willing to contract private insurances.

An insurance system such as Germany can have problems such as those mentioned by Smith and Petley (2009):

> *”Buildings insurance is only mandatory during the life of a mortgage and many householders – nobly tenants, pensioners and those in the lower socio-economic groups – either fail to insure or are under-insured” (Smith and Petley 2009).*

Moreover, this system does not encourage the owner to reduce the buildings vulnerability. According to Thieken et al. (2006), only 14 per cent of German insurers rewarded voluntary private mitigation measures.

As seen in different examples provided above, private or public insurance systems are characterized by certain advantages and disadvantages. However, most of the countries have not got a unique system, private or public, but a mix of both.

D.6 CONCLUSIONS

With their financial strength, insurance companies have the possibility to influence the economic losses due to natural hazards. This can be done either by reducing allowances, through incentives to reduce the vulnerability of properties, through research or by directly influencing the owner. The reduction of allowances to the policy-holder does not seem to be the most optimal way, because this benefits only the insurer and not the policy-holder.

Object vulnerability reduction will certainly be a challenge for the coming decades. With the current increase in damage costs and the prospect of an increase of natural disasters induced by global warming, many institutions will have to take into account the fragility of exposed objects. As such, insurance companies are concerned at the forefront.

By focusing on this research area, particularly through laboratory research or partnership with the scientific community, insurance companies seem to have anticipated this problem. Nevertheless, they still should go further: what has been written until now seems to confirm that this is the right way and that the insurance industry has everything to gain. Indeed, vulnerability of a given object has a huge impact on the final amount of damages.

However, vulnerability is not always taken into account by owners; though they may know the danger in which their property lies. Indeed, the systematic reimbursement does not encourage owners to take initiatives. Simple measures to reduce vulnerability could however be considered in most cases.

A lack of information seems to be the cause of this observation; reducing the vulnerability of a persons property is important and beneficial to decrease the amount of damage. Insurance companies can work in this direction.

According to Munichre (1997), motivation through financial incentives "*has already proved to be one of the most effective ways of encouraging the owner to take precautions. The best approach is to make sure that clients retain an adequate proportion of the risk themselves, especially by introducing substantial deductibles*". The role of insurance companies in natural hazards is financial, but must be used properly if it intends to reduce damages and not only the financial impact.

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