Engineering Measures for Landslide Disaster Mitigation

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Abstract

Correction of an existing landslide or the prevention of a pending landslide is a function of a reduction in the driving forces or an increase in the available resisting forces. Any remedial measure used must involve one or both of the above parameters.

According to IUGS WG/L, landslide remedial measures are arranged in four practical groups, namely: modification of slope geometry, drainage, retaining structures and internal slope reinforcement. This chapter discusses the planning and designing aspects of the landslide remedial measures in each group and presents some illustrative examples. In addition, debris flow mitigation measures are discussed in some detail. Back analysis of failed slopes is an effective tool for reliable design of the remedial measures while advanced numerical methods are nowadays frequently used to design safe and cost effective landslide remedial measures.

Selection of an appropriate remedial measure depends on: a) engineering feasibility, b) economic feasibility, c) legal/regulatory conformity, d) social acceptability, and e) environmental acceptability. There are a number of levels of effectiveness and levels of acceptability that may be applied in the use of these measures, for while one slide may require an immediate and absolute long-term correction, another may only require minimal control for a short period.

As many of the geological features, such as sheared discontinuities are not known in advance, it is more advantageous to put remedial measures in hand on a "design as you go basis". That is the design has to be flexible enough to accommodate changes during or subsequent to the construction of remedial works.

Keywords: Landslide disaster mitigation, Engineering measures, Debris flows, Back analysis, Numerical methods, Effectiveness and acceptability of remedial measures.

1. Introductory remarks

Landslides and related slope instability phenomena plague many parts of the world. Japan leads other nations in landslide severity with projected combined direct and indirect losses of $4 billion annually (Schuster, 1996). United States, Italy, and India follow Japan, with an estimated annual cost ranging between $1 billion to $2 billion. Landslide disasters are also common in developing countries and economical losses sometimes equal or exceed their gross national products (Sassa et al, 2005).

The paramount importance of landslide hazard management and mitigation is by and large recognized. Herein lies the guiding principle of the current chapter; i.e., to describe engineering methods to mitigate the landslide hazard associated risks in an appropriate and effective way.

2. Landslide disaster mitigation options

Risk mitigation is the final stage of the risk management process and provides the methodology of controlling the risk. At the end of the evaluation procedure, it is up to the client or policy makers to decide whether to accept the risk or not, or to decide that more detailed study is required. The landslide risk analyst can provide background data or normally acceptable limits as guidance to the decision maker but should not be making the decision. Part of the specialist's advice may be to identify the options and methods for treating the risk. Typical options would include (AGS, 2000):

- *Accept the risk* - this would usually require the risk to be considered to be within the acceptable or tolerable range.
- *Avoid the risk* - this would require abandonment of the project, seeking an alternative site or form of development such that the revised risk would be acceptable or tolerable.
- *Reduce the likelihood* - this would require stabilization measures to control the initiating circumstances, such as reprofiling the surface geometry, groundwater drainage, anchors, stabilizing structures or protective structures etc.
- *Reduce the consequences* - this would require provision of defensive stabilization measures, amelioration of the behavior of the hazard or relocation of the development to a more favorable location to achieve an acceptable or tolerable risk.
- *Monitoring and warning systems* - in some situations monitoring (such as by regular site visits, or by survey), and the establishment of warning systems may be used to manage the risk on an interim or permanent basis. Monitoring and warning systems may be regarded as another means of reducing the consequences.
- *Transfer the risk* - by requiring another authority to accept the risk or to compensate for the risk such as by insurance.
- *Postpone the decision* - if there is sufficient uncertainty, it may not be appropriate to make a decision on the data available. Further investigation or monitoring would be required to provide data for better evaluation of the risk

The relative costs and benefits of various options need to be considered so that the most cost effective solutions, consistent with the overall needs of the client, owner and regulator, can be identified. Combinations of options or alternatives may be appropriate, particularly where relatively large reductions in risk can be achieved for relatively small expenditure. Prioritization of alternative options is likely to assist with selection (Popescu, Zoghi, 2005).

3. Landslide disaster mitigation engineering measures

Correction of an existing landslide or the prevention of a pending landslide is a function of a reduction in the driving forces or an increase in the available resisting forces. Any remedial measure used must involve one or both of the above parameters.

IUGS WG/L (Popescu, 2001) has prepared a short list of landslide remedial measures arranged in four practical groups, namely: modification of slope geometry, drainage, retaining structures and internal slope reinforcement (Table 1). The flow diagram in Fig. 1 exhibits the sequence of various phases involved in the planning, design, construction and monitoring of remedial works (Kelly, Martin, 1986).

Hutchinson (1977) has indicated that drainage is the principal measure used in the repair of landslides, with modification of slope geometry the second most used method. These are also generally the least costly of the four major categories, which is obviously why they are the most used. The experience shows that while one remedial measure may be dominant, most landslide repairs involve the use of a combination of two or more of the major categories. For example, while restraint may be the principal measure used to correct a particular landslide, drainage and modification of slope geometry, to some degree and by necessity, are also utilized.

Modification of slope geometry is a most efficient method particularly in deep seated landslides. However, the success of corrective slope regrading (fill or cut) is determined not merely by size or shape of the alteration, but also by position on the slope. Hutchinson (1977) provides details of the "neutral line" method to assist in finding the best location to place a stabilizing fill or cut. There are some situations where this approach is not simple to adopt. These include long translational landslides where there is no apparent toe or crest. Also, situations where the geometry is determined by engineering constraints; and where the unstable area is and thus a change in topography, which improves the stability of one area may reduce the stability of another.

Drainage is often a crucial remedial measure due to the important role played by pore-water pressure in reducing shear strength. Because of its high stabilization efficiency in relation to cost, drainage of surface water and groundwater is the most widely used, and generally the most successful stabilization method. As a long-term solution, however, it suffers greatly because the drains must be maintained if they are to continue to function (Bromhead, 1992).

Surface water is diverted from unstable slopes by ditches and pipes. Drainage of the shallow groundwater is usually achieved by networks of trench drains. Drainage of the failure surfaces, on the other hand, is achieved by counterfort or deep drains which are trenches sunk into the ground to intersect the shear surface and extending below it. In the case of deep landslides, often the most effective way of lowering groundwater is to drive drainage tunnels into the intact material beneath the landslide. From this position, a series of upward - directed drainage holes can be drilled through the roof of the tunnel to drain the sole of the landslide. Alternatively, the tunnels can connect up a series of vertical wells sunk down from the ground surface. In instances where the groundwater is too deep to be reached by ordinary trench drains and where the landslide is too small to justify, an expensive drainage tunnel or gallery, bored sub-horizontal drains can be used. Another approach is to use a combination of vertical drainage wells linked to a system of sub-horizontal borehole drains. Fig. 2 presents pictures illustrating three of the most efficient drainage measures, namely sub-horizontal borehole drains, drainage wells and drainage tunnels (Japan Landslide Society, 2008).

Recent advances in the commonly used drainage systems include innovative means of drainage such as electro-osmotic dewatering, vacuum and siphon drains. In addition, buttress counterforts of course-grained materials placed at the toe of unstable slopes often are successful as a remedial measure. They are listed in Table 1 both under "Drainage" when used mainly for their hydrological effect and "Retaining Structures" when used mainly for their mechanical effect.

During the early part of the post-war period, landslides were generally seen to be "engineering problems" requiring "engineering solutions" involving correction by the use of structural techniques. This structural approach initially focused on retaining walls but has subsequently been diversified to include a wide range of more sophisticated techniques including passive piles and piers, cast-in-situ reinforced concrete walls and reinforced earth retaining structures. A schematic view of the commonly used retaining and slope reinforcement measures is given in Fig. 3 along with pictures illustrating two of these measures, namely large diameter caissons and ground anchors (Japan Landslide Society, 2008).

When properly designed and constructed, these structural solutions can be extremely valuable, especially in areas with high loss potential or in restricted sites. However fixation with structural solutions has in some cases resulted in the adoption of over-expensive measures that have proven to be less appropriate than alternative approaches involving slope geometry modification or drainage (DOE, 1994).

Over the last several decades, there has been a notable shift towards "soft engineering," non-structural solutions including classical methods such as drainage and modification of slope geometry but also some novel methods such as lime/cement stabilization, grouting or soil nailing. The cost of non-structural remedial measures is considerably lower when compared with the cost of structural solutions. On the other hand, structural solutions such as retaining walls involve opening the slope during construction and often require steep temporary cuts. Both these operations increase the risk of failure during construction for over-steeping or increased infiltration from rainfall. In contrast, the use of soil nailing as a non-structural solution to strengthen the slope avoids the need to open or alter the slope from its current condition.

Environmental considerations have increasingly become an important factor in the choice of suitable remedial measures, particularly issues such as visual intrusion in scenic areas or the impact on nature or geological conservation interests. An example of "soft engineering" solution, more compatible with the environment, is the stabilization of slopes by the combined use of vegetation and man-made structural elements working together in an integrated manner known as biotechnical slope stabilization (Fig. 4). The basic concepts of vegetative stabilization are not new; vegetation has a beneficial effect on slope stability by the processes of interception of rainfall, and transpiration of groundwater, thus maintaining drier soils and enabling some reduction in potential peak groundwater pressures. Except these hydrological effects, vegetation roots reinforce the soil, increasing soil shear strength while tree roots may anchor into firm strata, providing support to the upslope soil mantle through buttressing and arching. A small increase in soil cohesion induced by the roots has a major effect on shallow landslides. The mechanical effect of vegetation planting is not significant for deeper seated landslides, while the hydrological effect is beneficial for both shallow and deep landslides. However, vegetation may not always assist slope stability. Destabilizing forces may be generated by the weight of the vegetation acting as a surcharge and by wind forces on the vegetation exposed, though both these are very minor effects. Roots of vegetation may also act adversely by penetrating and dilating the joints of widely jointed rocks. The "Geotechnical Manual for Slopes" (Geotechnical Engineering Office of Hong Kong, 2000) includes useful information on the hydrological and mechanical effects of vegetation.

The concept of biotechnical slope stabilization is generally cost effective as compared to the use of structural elements alone; it increases environmental compatibility, and allows the use of local natural materials. Interstices of the retaining structure are planted with vegetation whose roots bind together the soil within and behind the structure. The stability of all types of retaining structures with open grid-work or tiered facings benefits from such vegetation.

4. Debris flow mitigation measures

Among debris flow mitigation measures, check dams are the most typical. Check dams in the stream capture debris flow directly and hold the sediment. Although check dams made of concrete are the most popular, some other types of check dams are also constructed for debris flow mitigation. Some relatively new types of check dams in Japan are presented in the following.

Concrete check dams (Fig.5) are the most popular. They are constructed not only to capture the runoff sediment directly, but also to decrease the volume and discharge runoff sediment. The latter function represents the so called 'sediment control function'.

The check dams filled with local sediment are used for construction cost reduction. This type of check dam is not made of concrete, but filled with sediments derived from the construction site and the inner sediment material is covered with steel walls. Steel walls are located in front of the dam and behind it (Fig.5). The construction of this type of dam can avoid the transport stage of the sediment from outside of the jobsite. It saves much time and cost during the construction stage.

Check dams with pitching logs (Fig.6) are sometimes used to harmonize the dam structure with the surrounding forest landscape. The dam body is made of concrete which provides enough strength against sediment and water discharge. Logs are used only for harmonizing with the landscape. This type of dam is generally constructed in a stream where the debris flow discharge is predicted to be very small and where a good looking landscape area like a national park is present. Logs derived by maintenance thinning of the forest are usually used for dam construction.

Check dams made of soil cement (Fig.7) are sometimes constructed in a stream with much sediment yielding from a large landslide located upstream of the jobsite or on a pyroclastic fan at the foot of a volcanic mountain. They also utilize local sediment. Usually local sediment generated by the construction of the dam (e.g. digging of the riverbed or cutting the slope for the inserting wing of the dam) should be transported away from the jobsite unless it is utilized for the construction. The transport of the sediment needs much time and cost. But check dams with soil cement avoid such transport process and reduce the cement quantity for the construction of dam body. This very much contributes to the reduction of the overall construction cost of the check dam. In some cases, the construction cost is reduced up to 30 % of the cost of the usual concrete dam in some volcanic areas of Japan. Soil cement is produced at the jobsite by agitating and mixing cement or cement milk with local sediment and other necessary materials at the jobsite. In Japan, two types of soil cement construction methods of check dams have been developed so far (Fig.8). The first one is ISM method. In this method, local sediment without large gravel is mixed and agitated with cement milk at the pit. The other method is called Sabo CSG or INSEM method. The sediment generated at the jobsite is mixed directly with cement milk and compacted at the jobsite by a vibration roller. The construction of check dams with soil cement in volcanic areas of Japan is often combined with 'Unmanned Construction System', abbreviated UCS, also called "construction robot" (Fig.9). In UCS, operation of the construction machine is conducted by remote control, so that the safety of workers is assured during the construction. At the foot of the active volcanic mountains, debris flow (lahar) often occurs even after a slight rainfall, so that it is difficult to keep safety of workers for the construction of check dams. But with UCS, workers have not always to work in the debris flow discharge area, so that the safety of the workers is very much improved. When UCS is adopted for the construction with soil cement, UCS machines are usually used for digging, transporting, mixing, agitating and filling sediment. In recent years, they are used even for the measurement of the dam body during construction in combination with GPS.

Open-type check dams (Fig.10) are popular and have been constructed at many sites for debris flow mitigation. Concrete slit dams and steel pipe grid dams are typical in Japan. Open type check dams allow sediment discharge to down stream, usually through the slit or open space, and sediment capture at the large scale flood and debris flow. The captured sediment is discharged to downstream little by little at the small scale flood. Therefore it is expected that the sediment pocket behind the dam body can be retrieved after the large flood or debris flow. This represents the sediment capture and retrieval function of this type of dam. In recent years, sediment discharge control function has also received attention. When there is a large discharge of water flow into the slit or grid, water back filling occurs because the cross-sectional area of flow suddenly decreases. As the water is accumulating behind the dam body, flow velocity largely decreases, so that sediment deposition occurs behind the dam body and the sediment discharge to downstream decreases. Concrete slit dams are expected to have these functions. On the contrary, steel pipe grid dam are expected to capture debris flow sediment while the captured sediment is discharged to downstream little by little after the debris flow. These repreent the debris flow capture function and the retrieval function of steel pipe grid dams.

5. Back analysis of failed slopes to design remedial measures

A slope failure can reasonably be considered as a full scale shear test capable to give a measure of the strength mobilized at failure along the slip surface. The back calculated shear strength parameters which are intended to be closely matched with the observed real-life performance of the slope, can then be used in further limit equilibrium analyses to design remedial works.

Shear strength parameters obtained by back analysis ensure more reliability than those obtained by laboratory or in-situ testing when used to design remedial measures.

In many cases, back analysis is an effective tool, and sometimes the only tool, for investigating the strength features of a soil deposit. However one has to be aware of the many pitfalls of the back analysis approach that involves a number of basic assumptions regarding soil homogeneity, slope and slip surface geometry and pore pressure conditions along the failure surface. A position of total confidence in all these assumptions is rarely if ever achieved.

Indeed, in some cases, because the large extension of a landslide, various soils with different properties are involved. In other cases the presence of cracks, joints, thin intercalations and anisotropies can control the geometry of the slip surface. Moreover progressive failure or softening resulting in strength reductions that are different from a point to the other, can render heterogeneous even deposits before homogeneous.

While the topographical profile can generally be determined with enough accuracy, the slip surface is almost always known in only few points and interpolations with a considerable degree of subjectivity are necessary. Errors in the position of the slip surface result in errors in back calculated shear strength parameters. If the slip surface used in back analysis is deeper than the actual one, c' is overestimated and ϕ' is underestimated and vice-versa.

The data concerning the pore pressure on the slip surface are generally few and imprecise. More exactly, the pore pressure at failure is almost always unknown. If the assumed pore pressures are higher than the actual ones, the shear strength is overestimated. As a consequence, a conservative assessment of the shear strength is obtainable only by underestimating the pore pressures.

Procedures to determine the magnitude of both shear strength parameters or the relationship between them by considering the position of the actual slip surface within a slope are discussed by Popescu and Yamagami (1994). The two unknowns - i.e. the shear strength parameters c' and ϕ' - can be simultaneously determined from the following two requirements (Fig. 11):

(a) F = 1 for the given failure surface. That means the back calculated strength parameters have to satisfy the c'-tan ϕ' limit equilibrium relationship;

(b) F = minimum for the given failure surface and the slope under consideration. That means the factors of safety for slip surfaces slightly inside and slightly outside the actual slip surface should be greater than one.

The fundamental problem involved is always one of data quality and consequently the back analysis approach must be applied with care and the results interpreted with caution.

Back analysis is of use only if the soil conditions at failure are unaffected by the failure. For example back calculated parameters for a first-time slide in stiff, overconsolidated clays could not be used to predict subsequent stability of the sliding mass, since the shear strength parameters will have been reduced to their residual values by the failure.

It is also to be pointed out that if the three-dimensional geometrical effects are important for the failed slope under consideration and a two-dimensional back analysis is performed, the back calculated shear strength will be too high and thus unsafe.

Although the principle of the back analysis method discussed above is correct, Duncan and Stark (1992) have shown that in practice, as a result of progressive failure and the fact that the position of the rupture surface may be controlled by strong or weak layers within the slope, the shear strength parameters cannot be uniquely determined through back analysis.

The alternative is to assume one of the shear strength parameters and determine the other one that corresponds to a factor of safety equal to unity. Duncan and Stark (1992) proposed to assume the value of ϕ', using previous information and good judgment, and to calculate the value of c' that corresponds to F=1. They recommended assume fully softened strength where no sliding has occurred previously, and residual strength where there has been sufficient relative shearing deformation along a pre-existing sliding surface.

Using the concept of limit equilibrium linear relationship c'-tan ϕ', the effect of any remedial measure (drainage, modification of slope geometry, restraining structures) can easily be evaluated by considering the intercepts of the c'-tan ϕ' lines for the failed slope (c_0', $\tan \phi_0'$) and for the same slope after installing some remedial works (c'_{nec}, $\tan \phi'_{nec}$), respectively. The safety factor of the stabilized slope is:

$$F = \min\left(F_c = \frac{c_0'}{c'_{nec}}, F_\phi = \frac{\tan \phi_0'}{\tan \phi'_{nec}} \right)$$

Errors included in back calculation of a given slope failure will be offset by applying the same results, in the form of c' - tan ϕ' relationship, to the design of remedial measures.

The above outlined procedure was used to design piles to stabilize landslides (Popescu, 2006) taking into account both driving and resisting force. The principle of the proposed approach is illustrated in Fig.12 which gives the driving and resisting force acting on each pile in a row as a function of the non-dimensional pile interval ratio B/D. The driving force, F_D, is the total horizontal force exerted by the sliding mass corresponding to a prescribed increase in the safety factor along the given failure surface. The resisting force, F_R, is the lateral force corresponding to soil yield, adjacent to piles, in the hatched area shown in Fig.12. F_D increases with the pile interval while F_R decreases with the same interval. The intersection point of the two curves which represent the two forces gives the pile interval ratio satisfying the equality between driving and resisting force.

The accurate estimation of the lateral force on pile is an important parameter for the stability analysis because its effects on both the pile-and slope stability are conflicting. That is, safe assumptions for the stability of slope are unsafe assumptions for the pile stability, and vice-versa. Consequently in order to obtain an economic and safe design it is necessary to avoid excessive safety factors.

6. Optimum planning and design of remedial measures by numerical analysis

Nowadays the budget for landslide disaster mitigation works in many countries is continuously shrinking due to economical restrictions. Therefore cost effective landslide mitigation measures are hardly needed.

This goal can be achieved by the optimum planning and design of landslide mitigation measures taking into account the actual landslide characteristics, adopting new construction methods or cheaper materials and reconsidering the construction process options. The following presents some general concepts on the optimum planning and design of landslide mitigation measures.

Numerical methods are largely used in the planning and design of landslide remedial measures. These include 3-D seepage analysis for the planning of drainage works and 3-D limit equilibrium slope stability analysis or 3-D deformation analysis by Finite Element Method (FEM) for the design of restraint works such as stabilizing piles or ground anchors.

Formerly, when our calculation ability was limited by the computer availability and capability, simplified 2-D numerical methods have been used for both seepage analyses and slope stability evaluations in order to design remedial measures such as slope geometry modification, drainage works, retaining structures or internal slope reinforcement structures. While it is apparent that the landslide processes are always 3-D, the 2-D seepage and slope stability analysis methods only treat longitudinal unit width sections of the landslide mass neglecting 3-D topographical and geological effects and 3-D pattern of the groundwater movement within the landslide mass which is also 3-D.

In recent years more sophisticated and more reliable, computer based, numerical analysis methods have been developed and adopted for the planning and design of the landslide mitigation works. These methods greatly contribute to a safer and more cost effective design of landslide mitigation works.

As stated above, the groundwater movement within the landslide mass is affected by the 3-D shape and geological structure of the landslide mass. In order to take into account these 3-D factors in the design of the drainage works, reliable information on the 3-D topography and geology of the landslide area is needed. The 3-D seepage modeling requires information not only on the site 3-D topography and geology but also on the 3-D distribution and variability of soil hydraulic and physical properties which govern groundwater movement. This type of analysis can more accurately simulate the movement of groundwater and therefore can result in optimum planning and design of the drainage works. It is to be noted that more information on site topography and geology and more geotechnical investigations to better define the variability of soil parameters are needed for a 3-D seepage analysis as compared with a 2-D analysis. However the additional cost associated with the supplementary investigations is compensated by the more reliable and cost effective design of the drainage works.

The above statement is valid also for 3-D slope stability analysis methods or 3-D deformation FEM analysis approaches when used for the design of the restraint works. 2-D slope stability analysis methods such as Fellenius or Bishop method, for circular failure surfaces, and Janbu or Spencer method, for non-circular failure surfaces, can be easily incorporated in simple computer programs or even used in hand calculations. However they do not reflect the 3-D landslide topography and geology and the 3-D variability of soil mechanical and physical properties. Where the shape of landslide mass is like a whisky-barrel, with the cross-sectional width and depth maximum at the center longitudinal profile and becoming smaller towards the lateral boundaries of the mass, we should consider in the general equilibrium of the sliding mass not only the resistant force at the toe but also at the lateral boundary resistant forces. In such a case only a 3-D analysis can adequately reflect the effect of the resistant forces in the cross-sectional direction. In addition, 3-D analysis can consider the resistant force of the anchor works which have an oblique direction in respect to the longitudinal section of the landslide mass. A 2-D analysis can not model appropriately oblique forces in respect to the longitudinal direction. As far as stabilizing piles are concerned, the 2-D analysis methods assume that the pile row acts as a wall providing a resistant force in longitudinal section. However the actual situation is clearly 3-D and the 3-D location of stabilizing pile and their interval along the cross-sectional direction should be considered for an appropriate planning and design of the stabilizing structures. If the pile interval is too large, the soil between piles can move down slope and therefore the stabilizing piles do not play their role though they may have enough structural resistance. Interaction between the piles and soil along cross-sectional direction should be considered in addition to the forces acting along longitudinal direction. 3-D FEM deformation analysis can adequately incorporate this effect.

7. Levels of effectiveness and acceptability that may be applied in the use of remedial measures

Terzaghi (1950) stated that, “if a slope has started to move, the means for stopping movement must be adapted to the processes which started the slide”. For example, if erosion is a causal process of the slide, an efficient remediation technique would involve armoring the slope against erosion, or removing the source of erosion. An erosive spring can be made non-erosive by either blanketing with filter materials or drying up the spring with horizontal drains etc.

The greatest benefit in understanding landslide-producing processes and mechanisms lies in the use of the above understanding to anticipate and devise measures to minimize and prevent major landslides. The term major should be underscored here because it is neither possible nor feasible, nor even desirable, to prevent all landslides. There are many examples of landslides that can be handled more effectively and at less cost after they occur. Landslide avoidance through selective location is obviously desired - even required - in many cases, but the dwindling number of safe and desirable construction sites may force more and more the use of landslide - susceptible terrain.

Selection of an appropriate remedial measure depends on: a) engineering feasibility, b) economic feasibility, c) legal/regulatory conformity, d) social acceptability, and e) environmental acceptability. A brief description of each method is presented herein:

a) Engineering feasibility involves analysis of geologic and hydrologic conditions at the site to ensure the physical effectiveness of the remedial measure. An often-overlooked aspect is making sure the design will not merely divert the problem elsewhere.

b) Economic feasibility takes into account the cost of the remedial action to the benefits it provides. These benefits include deferred maintenance, avoidance of damage including loss of life, and other tangible and intangible benefits.

c) Legal-regulatory conformity provides for the measure meeting local building codes, avoiding liability to other property owners, and related factors.

d) Social acceptability is the degree to which the remedial measure is acceptable to the community and neighbors. Some measures for a property owner may prevent further damage but be an unattractive eyesore to neighbors.

e) Environmental acceptability addresses the need for the remedial measure to not adversely affect the environment. De-watering a slope to the extent it no longer supports a unique plant community may not be environmentally acceptable solution.

Just as there are a number of available remedial measures, so are there a number of levels of effectiveness and levels of acceptability that may be applied in the use of these measures. We may have a landslide, for example, that we simply choose to live with; one that poses no significant hazard to the public, whereas it will require periodic maintenance for example, through removal, due to occasional encroachment onto the shoulder of a roadway.

Most landslides, however, must usually be dealt with sooner or later. How they are handled depends on the processes that prepared and precipitated the movement, the landslide type, the kinds of materials involved, the size and location of the landslide, the place or components affected by or the situation created as a result of the landslide, available resources, etc. The technical solution must be in harmony with the natural system, otherwise the remedial work will be either short lived or excessively expensive. In fact, landslides are so varied in type and size, and in most instances, so dependent upon special local circumstances, that for a given landslide problem there is more than one method of prevention or correction that can be successfully applied. The success of each measure depends, to a large extent, on the degree to which the specific soil and groundwater conditions are prudently recognized in an investigation and incorporated in design.

As many of the geological features, such as sheared discontinuities are not known in advance, it is more advantageous to put remedial measures in hand on a "design as you go basis". That is the design has to be flexible enough to accommodate changes during or subsequent to the construction of remedial works.

8. Invited presentations

8.1 The Forest City Landslide, South Dakota, USA (by Vernon R. Schaefer)

Following inundation of the Oahe Reservoir in the 1960s in South Dakota, USA, numerous landslides developed along the reservoir rim. A particularly large landslide reactivated at the location where U.S Highway 212 crosses the reservoir. The landslide is locally known as the Forest City landslide, after the former village that occupied the location prior to reservoir impoundment. The U.S. Highway 212 Bridge was constructed prior to closure of the Oahe Dam, as a replacement to a bridge crossing the Missouri River some 10 km upstream. Unbeknownst to the bridge designers, the bridge was located at the toe of an ancient landslide. Rising reservoir levels caused reactivation of the landslide; however it took many years before this was recognized. Water levels began rising in the Oahe Reservoir in the early 1960s and the first bridge distress was noted in 1962. By 1965 an expansion device at the bridge abutment had closed. The first recognition of geotechnical problems was in about 1968 and extensive monitoring of the landslide at the site began in 1972. Throughout the 1970s and 1980s intermittent monitoring and movements occurred, with occasional concerns for safety of the bridge. Hazard warning devices were installed at the bridge abutment in case of failure and loss of the traffic lanes. Continuing movements brought the need and realization for stabilization to the attention of political officials. Extensive remedial investigations began in 1988, with construction of several stabilization techniques during the 1990s, including stone columns, unloading of the driving force and installation of shear pins in the toe of the slide.

The local importance of the Highway 212 bridge structure stems from the fact that it is the only reservoir crossing for some 140 km upstream and downstream of the bridge. The Forest City landslide measures approximately 1.7 km wide, 1.25 km long from head to toe, and 125 meters high from head to toe. The depth of the sliding surface is from 60 to 120 meters below ground surface. It has been estimated that the landslide involves on the order of 75 million cubic meters of soil and rock debris and covers over three square kilometers of land area.

The stratigraphy at the site consists of firm and weathered Pierre Shale overlain by glacial till materials consisting of a heterogeneous clay matrix with sand, silt and clay and numerous gravel beds, likely the result of erosion in geological time. Fill was placed at the toe of the slope to provide an abutment for the bridge. Extensive instrumentation was placed at the site to allow determination of the location of the failure plane and reasonable determination of water levels in the shale and overlying glacial till materials. The instrumentation showed that the movements were occurring at or just above the contact of the weathered shale and the firm shale. Water level measurements indicated a complex system between the shale, glacial till and reservoir levels. Laboratory testing of the Pierre Shale and back analyses of the failure indicated that the residual strength of the weathered shale was in the range of 6 to 8 degrees.

The remedial measures (Fig.13) were constructed in three phases. The first phase completed in 1993 consisted of drilling large diameter stone columns into the toe of the slide to stabilize local slides near the bridge abutment. The second phase completed in 1995 and 1996 consisted of excavating a deep corridor through the center of the overall sliding mass to unload the driving forces. The corridor was approximately 185 meters wide, some 600 meters in length and was made some 40 meters into glacial till materials. Approximately 7 million cubic meters of material was removed from the cut. The third phase was completed in 1998 and consisted of the placement of 66 shear pins at the toe of the slide, to depths of 45 meters below ground surface to intersect deep shear planes in the overall sliding mass. The shear pins were one meter by three meter rectangular reinforced concrete members. Fig. 13 shows construction of the shear pins.

Prior to the remedial measures the movements of the main slide and local slides ranged from 100 mm to 250 mm per year. The remedial measures have reduced the movements to less than 2 mm per year and have been performing well for nearly a decade.

8.2 Back Analysis of Landslides to Allow the Design of Cost-Effective Mitigation Measures (by S.R. Hencher, S.G. Lee, and A.W. Malone)

This paper concerns forensic studies of landslides and the improved understanding that can result from such studies, both for local mitigation and to allow more rational management of slopes. The true mechanism of a landslide is often difficult to unravel - there may be many contributing factors and the investigator needs to act as a detective, looking for evidence, developing theories and testing these through further observation, analysis and focused investigation. One of the key questions is often why a landslide has occurred at a particular location, at a particular time (especially where there is no immediate trigger) and with a particular geometry rather than elsewhere in the same slope or in adjacent slopes. It is noted that the very nature of landslides calls for experience in geomorphology, engineering geology, structural geology, hydrogeology and soil and rock mechanics in forensic investigation. Without such knowledge within the investigating team, landslides may well be misinterpreted with incorrect, simple explanations offered for what is fundamentally more complex. Reference is made to examples where different teams have given different explanations for the same landslide event and this is used to emphasis the need for a balanced investigation team.

In many cases, a fundamentally important aspect is adverse and often complex geology and hydrogeology and this is illustrated for several rock and deeply weathered slopes. The presentation is illustrated with reference to detailed investigations of landslides in Hong Kong, Korea, Malaysia and Europe. Cases include large scale rock failures and weathered rock examples from Hong Kong, the current massive landslide at Pos Selim in Malaysia (Fig.14) and a fatal landslide in South Korea. Other examples are used to illustrate how landslides can be interpreted to gain a better understanding of mass shear strength, strain-controlled shear strength and hydrogeological controls.

Failures in engineered slopes are often particularly revealing in that they demonstrate flaws in thinking, investigation and analysis from which lessons can be learned. Examples include failed slopes that had been investigated using standard ground investigation and instrumentation techniques but where the true mechanism had been overlooked or missed.

It is acknowledged that whilst it is relatively easy to identify the key aspects of a landslide after the event, it is a much more difficult task to use that interpretation to make predictions regarding the hazard levels in other slopes. Examples are given of where such lessons have been used to reassess other slopes and to make decisions regarding the need for landslide mitigation works.

In terms of mitigation, it is very important that an ongoing, progressive landslide is properly understood to ensure that correct and cost effective mitigation measures are adopted. Monitoring is important for assessing an ongoing landslide risk but that monitoring must to be linked to models, identified through proper investigation and analysis that can then be tested through prediction and measurement.

8.3 Soft Engineering and Drainages for Slope Stabilization: Ground Water Control and Vegetative Bio Techniques (by G. Urciuoli and R. Papa)

Introduction: Soft engineering, also known as bioengineering, is a technique which uses native plant materials to stabilize eroding slopes and shallow landslides. This technique can give good results if it is jointed to other types of control works, as drainages of surface and subsurface water, constituting a more natural and less invasive method for soil stabilization, respect to “hard” engineering techniques (e.g. walls, piles).

Drainage: In saturated soils, drains are widely used as control works against slope instability, as they are less costly than other types of control works and are suitable for a large number of cases, even when the landslide is very deep and structural measures are inadequate. The mechanical role of drains inside slopes consists in a decrease in pore pressures in the subsoil and consequently of an increase in effective stresses and soil shear strength in the whole drained domain. In particular, increase in shear strength along the active or potential sliding surface, at the base of the landslide body, is responsible for slope stability improvement due to the work of drains. Therefore the first step in the design of a drainage system consists of determining a distribution of pore pressure changes, for which the factor of stability is increased up to the value chosen by the designer.

The next step consists of designing the geometric configuration of drains that determines the distribution of pore pressure previously calculated by means of slope analysis.

The effect of the drainage system is usually analyzed in steady-state condition, which is attained some time after drainage construction (in the long term). The analysis is carried out by considering continuously present, at the ground surface, a film of water, able to recharge the water table. In the literature, results of steady-state analysis are represented in non-dimensional design charts that technicians generally use to project drainage systems.

After drain excavation, a transient phenomenon of equalization of pore pressures occurs, provoking subsidence of the ground surface, whose intensity depends on: i) compressibility of the soils concerned, ii) thickness of the drained domain, iii) lowering of the water table. Therefore problems related to excessive settlements are expected when the drained soil is very thick, as in the case of deep drains.

As regards the transient phase, two aspects have to be evaluated in the design:

- whether the delay until the complete efficiency of drains is compatible with the destination of the area,
- whether associated settlements can damage buildings and infrastructures at ground surface.

The water flow captured and discharged by drains depends strongly on the permeability of the drained soils. In steady-state condition the permeability does not affect the lowering of the pore pressures in the subsoil, which depends on the hydraulic conditions at the boundaries of the examined domain and the geometry of the drainage system. Thus the quantity of water discharged is not an indicator of the good working of drains, which must be investigated by means of piezometers, to measure the level of the water table as modified by drains. Indeed, pore pressure changes are the most direct and useful indicators of drains in good working order. Measurements of superficial and deep displacements are good indicators of overall slope stability and can be carried out to complete the framework of information obtained from piezometer surveys.

The role of vegetation: Also hydraulic interaction between the subsoil and the atmosphere and the role of vegetation on this phenomenon have been analysed, to predict soil water content and suction regime in the subsoil, as a function of meteoric and seasonal variations. Important differences in soil water content have been expected during dry seasons when evapo-transpiration is high (due to high temperature and direct sun radiation on soil surface) and there are leaves on the branches of trees.

During humid seasons suction in the subsoil decreases and assumes more or less the same values where there are trees and where vegetation is absent or has been cut.

8.4 Stabilization of Slopes: An Experience from Norway (by V Thakur, A Watn, S Christensen, E Øiseth, S Nordal, G Priol and K Senneset)

Over the last couple of decades the use of deep soil stabilization with dry mixing of lime-cement (Fig.15) has increased in Norway. Especially lime-cement columns located in ribs for slope stabilization have been used on several occasions. The method has proved much cost competitive in comparison to traditional solutions. Unfortunately some failures of lime-cement stabilized slopes have occurred, putting focus on the relevance of the parameters for strength and deformation of both the undisturbed and the stabilized soil. Existing methods for in situ and laboratory testing are generally related to total strength parameters. A principle for design using effective strength parameters is believed to give a more realistic representation of the real strength of the lime-cement ribs and the interaction with the surrounding soil. The inhomogeneous nature of the stabilized soil thus requires a relatively large amount of data to establish relevant strength and deformation parameters. A real case of lime cement stabilization is presented in this paper. Also a numerical analysis based on effective strength parameters has been conducted and has given a more realistic representation of the real strength of the lime-cement ribs and the interaction with the surrounding soil. This modelling has been compared to a 3D FEM model using the Plaxis code in order to permit a better evaluation of the reinforcement produced by the ribs. This work constitutes the first step for the development of a new design principle for this kind of complex geotechnical structure. The background for this article is based on the experience of SINTEF and NTNU in projects connected to slope stabilization and embankments, and through involvement and guidance in academic theses at NTNU.

8.5 Modeling the Behavior of Rockfall Protection Fences (by Cantarelli G.C., Giani G.P., Gottardi G. & Govoni L.)

The paper presents an analytical model for a rock block impacting rockfall protection barrier metallic nets (Fig.16). The model can be used to evaluate the net elongation and its braking time. The analytical model can also determine when an impact is exclusively elastic or when the barrier provides a plastic response.

The analytical procedure has been calibrated through a comparison between the computed prediction and the results of full scale impact tests carried out on several blocks of different size, following the instructions provided by the Guideline for European Technical Approval of Falling Rock Protection Kits (ETAG).

The results presented in the paper concern the experiments carried out on a vertical-drop test site, which is a structure able to accelerate a concrete block to an established speed and to impact it, in free-fall motion, onto a sample of rockfall protection barrier. The sample of the barrier is made of three functional modules (i.e. three fence segments) and is anchored orthogonally to a vertical slope. A crane is used to handle the testing block made out of polyhedron shaped concrete. During the impact test, the block trajectory is vertical and the block impacts into the centre of the middle functional module. No ground contacts occurs before the impact, ensuring that there is no energy loss except the air friction energy loss. Therefore the kinetic energy is a sole function of the mass and falling height of the block. As a result this kind of test is particularly suitable for the purposes of model calibration. The test site is also provided with high-definition video cameras for the direct measurement of the net maximum elongation and braking time in dynamic conditions. These measured values, when evaluated analytically, show a remarkably good agreement with the experimental results.

The metallic net behaviour is assumed elastic to describe the block arrest and the phenomenon is described by a non-homogeneous constant coefficient second order differential equation. The constant coefficients are determined by imposing the initial conditions. The motion equation in the post impact phase is obtained by integrating the net behaviour differential equation. Thus the net maximum elongation and the arrest time are determined.

The analytical model can be applied to other cases of blocks of different geometry impacting onto metallic nets. The results of these applications put into evidence how a barrier design based only on energetic criteria might not be suitable.

A final discussion shows how the analytical model proposed in this paper can also be used for debris flow or snow avalanche protection works.

8.6 An evaluation method of landslide prevention works in Yuzurihara Landslide (by Nobuaki KATO, Ryosuke TUNAKI, keiji MUKAI, Kazuyuki SATO, Takumi YOSHIZAWA)

The presentation reports an evaluation method of landslide prevention works in Yuzurihara landslide.

Yuzurihara Landslide is located on about 20km south from Takasaki City, Gunma prefecture, Japan (36°08'N; 139°02'E). It faces the southwest side of a mountain ridge and has the designated landslide prevention area of 600m long 1700m wide and 40-50m deep (Fig.17). There are about forty residences on the landslide area and the route 462 passes in the midst of the landslide. The basement rock of the landslide is crystalline shist belonging to the Sambagawa Belt.

The landslide became activated in 1910, 1938, and 1947. By 1969, ground water drainage works taken by Gunma prefectural government inactivated the landslide. In 1991, the landslide reactivated due to rainfall, which damaged the route 462 and residential structures. Afterwards, the landslide prevention works become large-scale, and it was replaced under the direct control of the Ministry of Construction in 1995.

The prevention area is divided into three areas, Kayakabu-Karyu, Kayakabu-Joryu and Shimokubo. In Kayakabu-Karyu area, nine units of drainage boring works and fourteen water catchment wells have been implemented by the prefectural government, and two units of drainage boring works, six water catchment wells and two drainage tunnels (683m and 541m long) have been implemented by the central government (Fig.17). Control works have been completed in Kayakabu-Karyu area and Kayakabu-Joryu area. On the other hand, in Shimokubo area, the control works are now under construction.

In order to evaluate the effectiveness of the landslide prevention works, observations and stability analyses are executed in each year. Ground movement in the area is measured by GPS, extensometers, inclinometers and borehole type tilting meters, and ground water level is measured by groundwater level gauges.

Kayakabu-Karyu area is composed of a large block (Block I), and five small-scale blocks are located downward of the large block. The results of measurement show that the Block I and small-scale blocks have become non-active sinse the control works were carried out.

In Kayakabu-Karyu area, the safety factor of Block I is calculated by three dimensional analysis, and small-scale blocks are calculated by two dimensional analysis. The three dimensional stability analyses are executed under the conditions of two types of groundwater level, measured groundwater levels and simulated groundwater levels. To simulate the groundwater level at the return period of 100 years, a three dimensional saturated-unsaturated finite element groundwater seepage analysis model have been developed. The result of the stability analyses shows that the safety factor of Block I exceeds 1.10 at high water level of recent years, and was 1.03 at the return period of 100 years rainfall. On the other hand, at four small-scale blocks the safety factors fall below 1.00 at the return period of 100 years rainfall. The results of the stability analyses are consistent with the result of measurements which shows that the landslide in Kayakabu-Karyu area is generally non-active.

To optimize public investments, it is significant to carry out precise evaluation on effectiveness of landslide prevention work. In Japan, the budget for landslide preventions is decreasing and needs for accurate evaluating technique for landslide stability is growing. The authors are convinced that in Kayakabu-Karyu area, the three dimensional groundwater seepage analysis and stability analysis enabled to calculate accurate safety factor and to prevent overinvestment on the landslide prevention works.

8.7 Optimum Design of Landslide Stabilizing Piles by Centrifugal Loading Experiments and FEM (by Yasuo ISHII, Kazunori Fujisawa, Yuichi UENO, Yuichi NAKASHIMA, Keiichi ITO)

Pile works are one of the useful structural countermeasures against landslides, which are constructed to connect the movable landslide mass and the stable ground with steel pipe to restrain the movement.

In an effective plan of stabilizing piles, optimum pile design such as its position and intervals is desired in order to obtain high reliability and low construction cost of pile works.

In Japan, intervals of piles have been designed according to landslide depth under individual experience up to the present (Table 2).

Therefore the author estimates optimum design of landslide stabilizing piles by centrifugal loading experiments (Fig.18) and their back analyses by FEM under changes of some geo-condition such as strength, ductility and so on.

This estimation shows that the intervals of piles can be changed due to geo-condition, moreover maximum intervals of piles can be less than eight (8) times of pile diameter.

According to these experiments and numerical analysis, optimum design of landslide stabilizing piles could be established. Moreover, the 3D slope stability analysis under reasonable pile works followed by the author, which resulted in moderation of landslides and reduction of the pile work costs.

8.8 Mitigation of Earthquake Triggered Landslide in Sri Lanka –A Myth or Reality? (by S.B.S. Abayakoon)

Sri Lanka is a pear shaped island located just below the southern tip of India within the rectangle bounded by 79.7E-81.8E and 5.9-9.8N. General topography of the island can be described by three peneplanes cut into a rocky framework rising from the sea. The highest peneplane of elevation 1500-2500 m above Mean Sea Level (MSL) is completely surrounded by the middle peneplane of elevation over 900 m. The lowest coastal peneplane which is of average height less than 100 m is generally flat and sometimes gently undulating. The central highlands start from an elevation of about 270 m above MSL and comprise of nearly 22% of the total land area covered with hilly or mountainous terrain, embracing well over one million hectares, spread over seven districts.

The central region of Sri Lanka is hilly and mountainous with bedrock overlain by residual soils and colluvium. The occurrence of fresh landslides and reactivation of dormant landslides is a frequent phenomenon in this area during heavy rainy periods. These slides cause severe damage to life and property and therefore considered as the most significant natural hazard in Sri Lanka. It must be recorded that although there were about 40,000 casualties due to the Boxing Day Tsunami disaster of 2004, the possibility of reoccurrence of such disasters is quite remote. On the other hand, floods are quite common occurrences in Sri Lanka, but the damages to life and property due to floods are quite small compared to those due to landslides.

Policymakers and researchers have identified landslides as one of the major area that needs attention when one considers natural disasters of Sri Lanka. These have lead to development of landslide hazard maps and identification of traditional areas that are considered as vulnerable to future landslides. Recently, however, there has been an increase in the occurrence of landslides in areas away from those that are considered as landslide prone.

Traditionally, seismicity in Sri Lanka has not been considered important although it has been discussed by a few authors at various forums (Fernando and Kulasinghe, 1986, Vitanage, 1995, Wimalaratne, 1993, Fernando, 1982, Abayakoon, 1996, 1998, 2000, 2001). However, the Boxing Day Tsunami disaster of 2004, that was a direct result of an earthquake occurred near the Island of Sumatra, a new awareness has been developed in this area of research. The said earthquake, measuring over 9.0 in Richter scale of Magnitude is supposed to have resulted in creating a new fault line within a few hundred kilometers of the southern coast of Sri Lanka.

Increase in the number of small scale landslides and the occurrence of new landslides in areas that have not been considered landslide prone, suggests that there may have been a connection between increased seismicity of the area, and landslide occurrences of the island. A study of the pattern of landslides before and after the Tsunami and ways and means of mitigating such occurrences in the future are now considered important and essential.

This paper describes some recent advances made in landslide research in Sri Lanka with special emphasis on effect of seismicity. As seismic events cannot be forecasted, mitigation against seismic vulnerability of landslides is not an easy task. However, some approaches that can be adopted are also discussed in detail. The paper also suggests possible directions in which the research can be advanced with modern day sophisticated analysis procedures.

9. Concluding remarks

Much progress has been made in developing techniques to minimize the impact of landslides, although new, more efficient, quicker and cheaper methods could well emerge in the future. There are a number of levels of effectiveness and levels of acceptability that may be applied in the use of these measures, for while one slide may require an immediate and absolute long-term correction, another may only require minimal control for a short period.

Whatever the measure chosen, and whatever the level of effectiveness required, the geotechnical engineer and engineering geologist have to combine their talents and energies to solve the problem. Solving landslide related problems is changing from what has been predominantly an art to what may be termed an art-science. The continual collaboration and sharing of experience by engineers and geologists will no doubt move the field as a whole closer toward the science end of the art-science spectrum than it is at present.

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Table 1 Short list for landslide remedial measures arranged in four practical groups: modification of slope geometry, drainage, retaining structures and internal slope reinforcement.

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 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 filled with free-draining geomaterials (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. Vacuum dewatering
2.9. Drainage by siphoning
2.10. Electroosmotic dewatering
2.11. 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 earth retaining 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. Heat treatment
4.8. Freezing
4.9. Electroosmotic anchors
4.10. Vegetation planting (root strength mechanical effect)

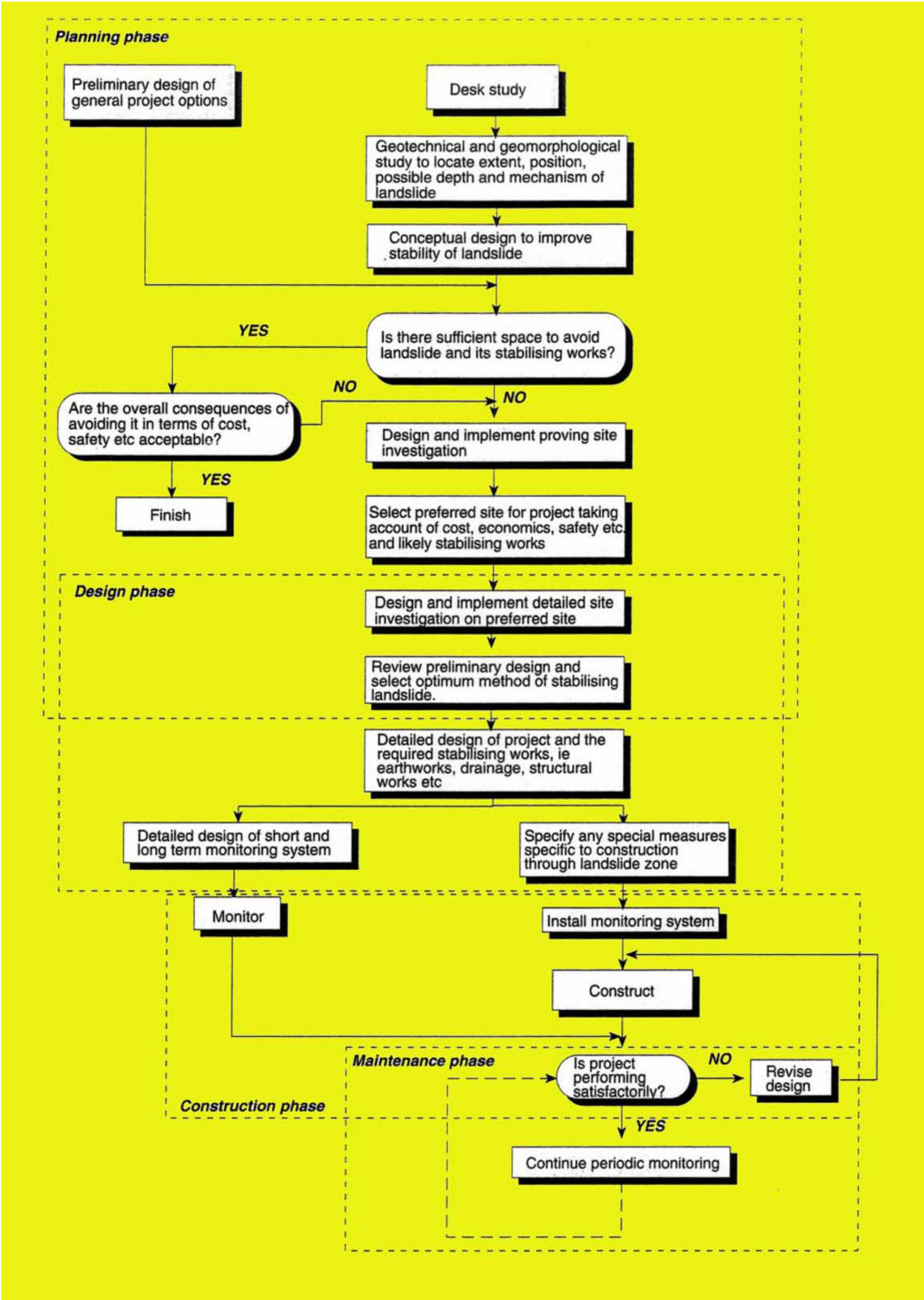

Fig.1 Flow diagram showing the sequence of various phases involved in the planning, design, construction and monitoring of remedial measures

Fig.2 Comprehensive drainage measures: Sub-horizontal Drainage Boreholes (upper). Vertical Drainage Wells (lower left). Drainage Tunnels (lower right). Both drainage wells and drainage tunnels are associated with numerous sub-horizontal and sub-vertical boreholes for groundwater collection.

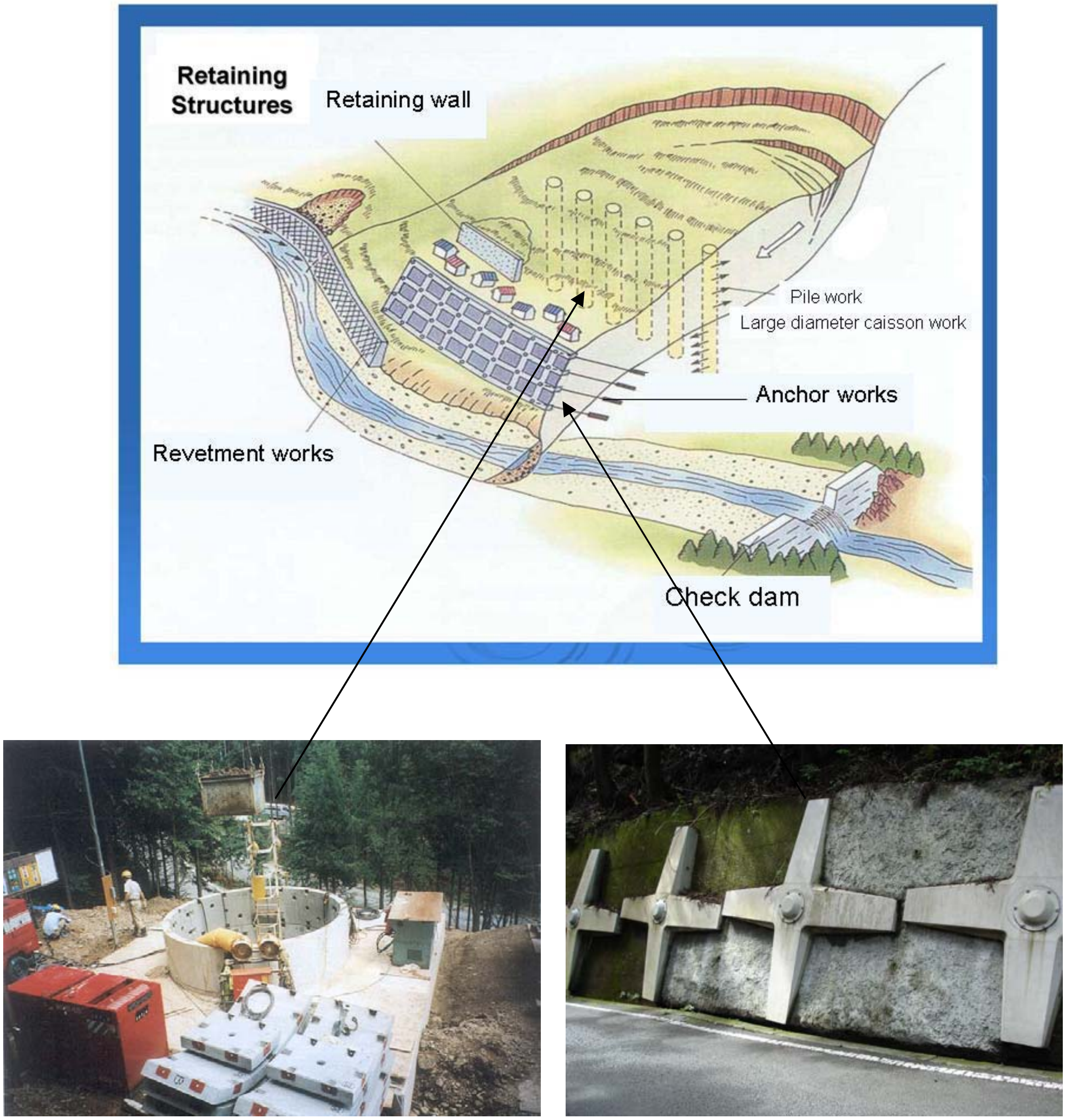

Fig.3 Schematic view of the commonly used retaining and slope reinforcement measures (upper) along with pictures illustrating two of these measures: Large Diameter Caissons (lower left) and Ground Anchors (lower right).

Fig.4 Biotechnical slope stabilization: Combined use of vegetation and man-made structural elements working together in an integrated manner (upper). Vegetation planting combined with rock counterfort construction (lower). Vegetation has both hydrological and mechanical beneficial effects.

Fig.5 Concrete Check Dam (upper): concrete panels are pitched in the front of the dam body in order to harmonize with the surrounding landscape. Check Dam Filled with Local Sediment (lower): steel panels cover the dam body consisting of sediment.

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Fig.6 Check Dam with Pitching Logs: pitching logs in front of the dam body harmonize with the surrounding forest landscape.

Fig.7 Check Dam Made of Soil Cement: the wings are made of soil cement and the spillway is made of concrete.

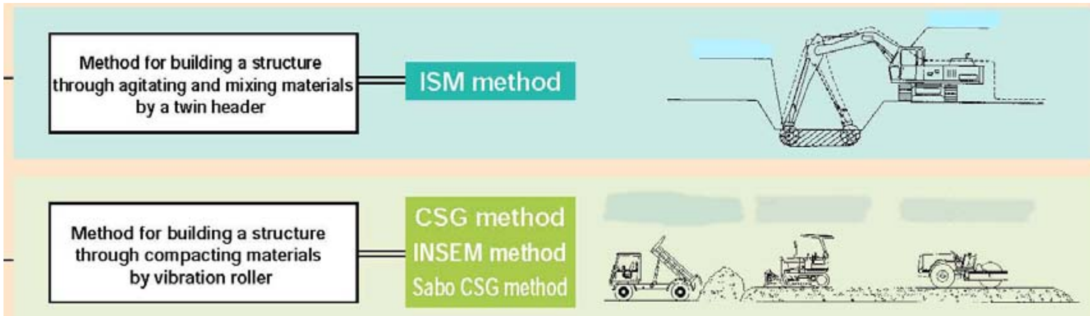

Fig.8 Soil Cement Construction Methods. They are divided into two categories: the first one is the ISM method and the other includes CSG, INSEM and Sabo CSG method.

Fig.9 Backhoe operated by Unmanned Construction System (UCS). Robot arm and surveying camera are set at the operating location. Operator remotely controls the machine by the robot arm and camera.

Fig.10 Open-type Check Dam: The Concrete Slit Dam (upper) is expected to have both sediment capture and retrieval functions as well as sediment discharge control function. The Steel Pipe Grid Dam is expected to catch the debris flow and discharge the captured sediment after the debris flow.

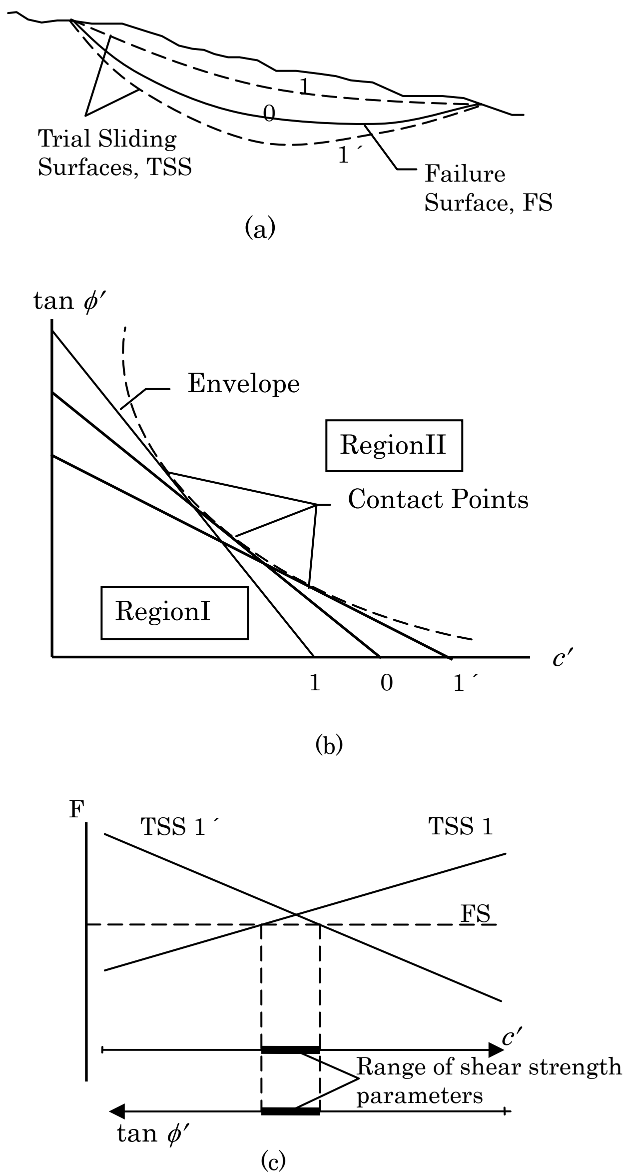

Fig.11 Back Calculation of the Shear Strength Parameters from Slope Failures.
The two unknowns (c' and ϕ') can be simultaneously determined from the following two equations involving the slope safety factor: F=1 and F=minimum.

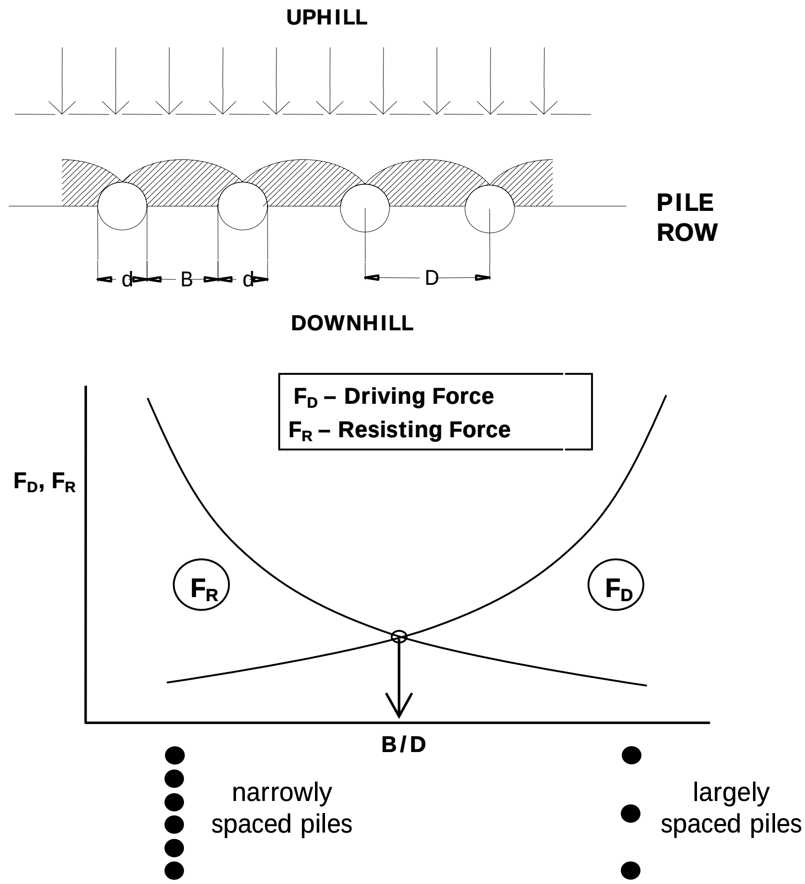

Fig.12 Design of Piles to Stabilize Landslides. Both driving and resisting force acting on each pile in a row should be considered to derive the optimum non-dimensional pile interval ratio B/D.

Fig.13 Remedial Measures of the Deep Seated Forest City Landslide: Excavator used to construct the one meter by three meters shear pins (left). Reinforcing cage being lowered into the excavation (right).

Fig.14 Pos Selim Landslide in Malaysia. Detailed investigations of the complex geology and hydrogeology conditions have been performed to define the landslide mechanisms.

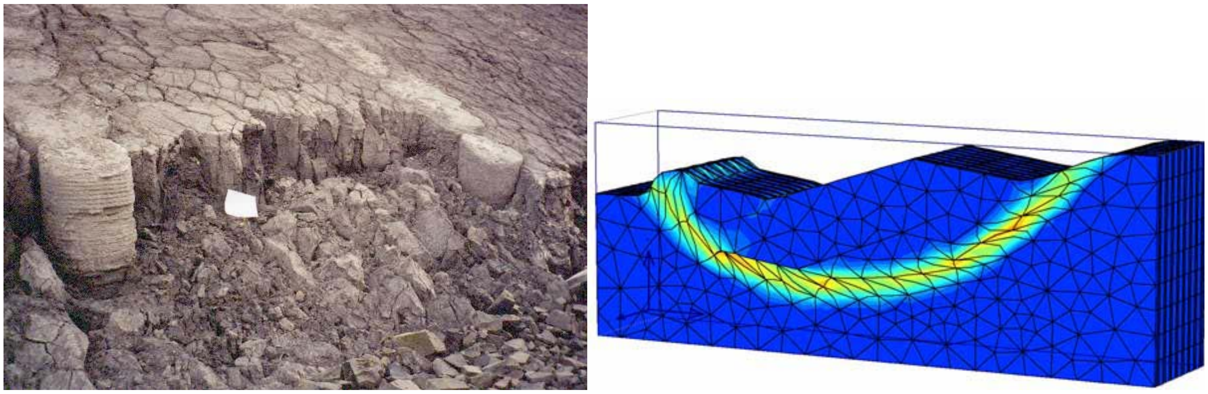

Fig.15 Soil Stabilization Using Lime-Cement Piles (left); Three Dimensional Finite Element Modelling of the interaction between the reinforcing lime-cement ribs and surrounding soil (right).

Fig.16 Full Scale Modeling of the Behavior of the Rockfall Protection Fences: Rock blocks of different sizes are dropped vertically onto the protection net to assess its deformation and breaking time.

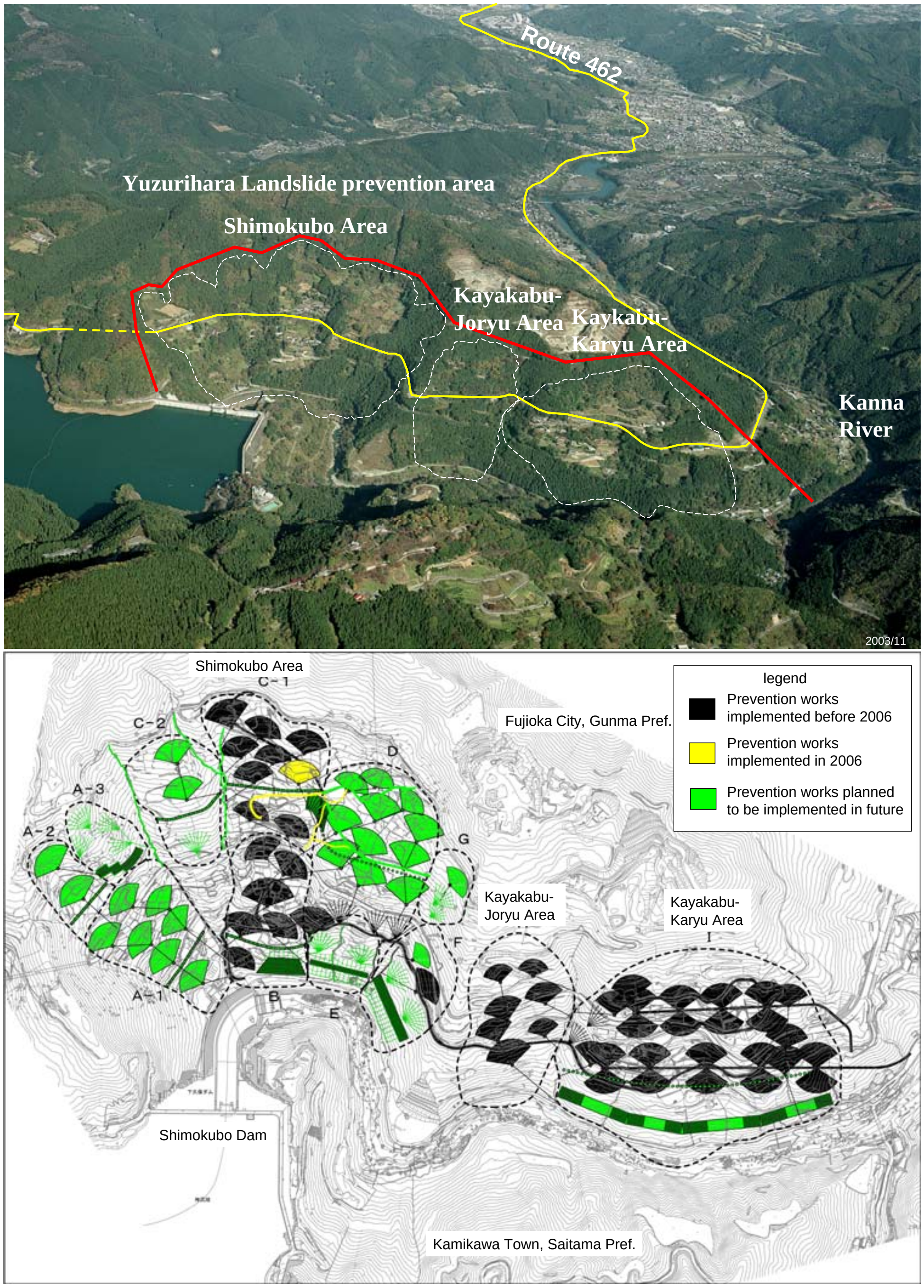

Fig.17 Yuzurihara Landslide, Japan: Landslide area comprising several blocks (upper). Map of the mitigation works consisting mainly of large diameter vertical drainage wells and sub-horizontal drainage boreholes (lower).

Landslide Depth (m)	Standard Interval of Piles (m)
< 10	≦ 2.0
10 ～ 20	≦ 3.0
20 ≦	≦ 4.0

Table 2 Japanese Practice for Determining Stabilizing Piles Interval Based on Landslide Depth.

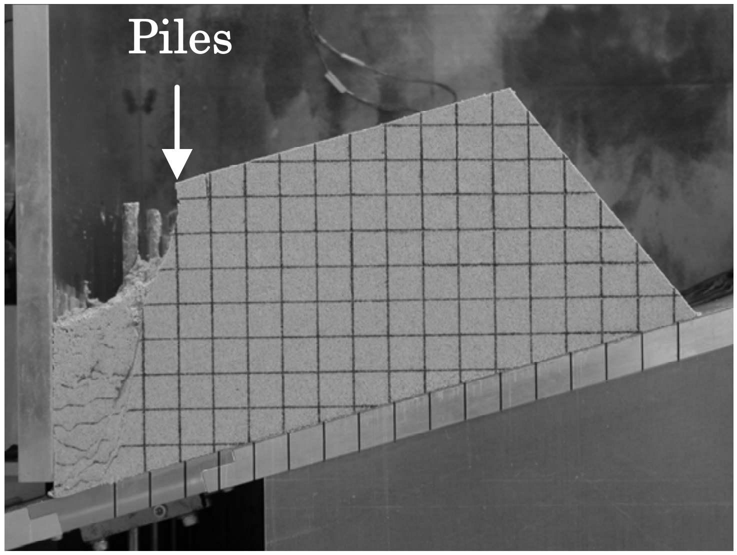

Fig.18 Slope Condition after the Centrifugal Loading Experiment: Slope failure occurred around the piles inserted into the slope.