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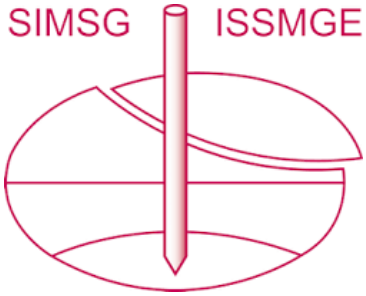

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General report/Discussion session 21: Control of landslides and instrumentation

Rapport de spécialistes/Séance de discussion 21: Contrôle des glissements et instrumentation

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INTRODUCTION

The fifteen papers falling in the theme of Session 21 on **Control of Landslide & Instrumentation** are dealt with in this **General Report** as classified in Fig. 1.

INSTRUMENTATION AND FIELD MONITORING

The State-of-the-Art reportings on Instrumentation and Monitoring of Landslides and other Mass Movements have become almost a regular feature, Bhandari (1980), (1984); Pilot (1984); Bhandari (1988). The level of instrumentation technology, reliability & accuracy of measurement, and speed of data processing have touched a new high but perhaps the same could not be said of dependability on interpretation of field observations, which continue to dodge agreement between prediction and performance.

One of the most common, useful and economical approach to monitor the performance of a slope is through measurement of it's surface and subslope movements or displacements. The measurements made in professional practice are usually intermittent rather than continuous in nature, interalia, for reason of economy. Bhandari (1988) considered in detail the implications to establish that intermittancy of measurement, in many cases, may lead to deceptive prediction of a landslide or slope failure. This is because such measurements can never tell the complete tale, for the frequency of displacement measurements are not usually high enough to cover the frequency of occurrence of motion events. Vital history of movement, including abrupt increase of displacement rates, surges etc. are missed if intervals between observations are too long. Predictions of displacement rates from the known

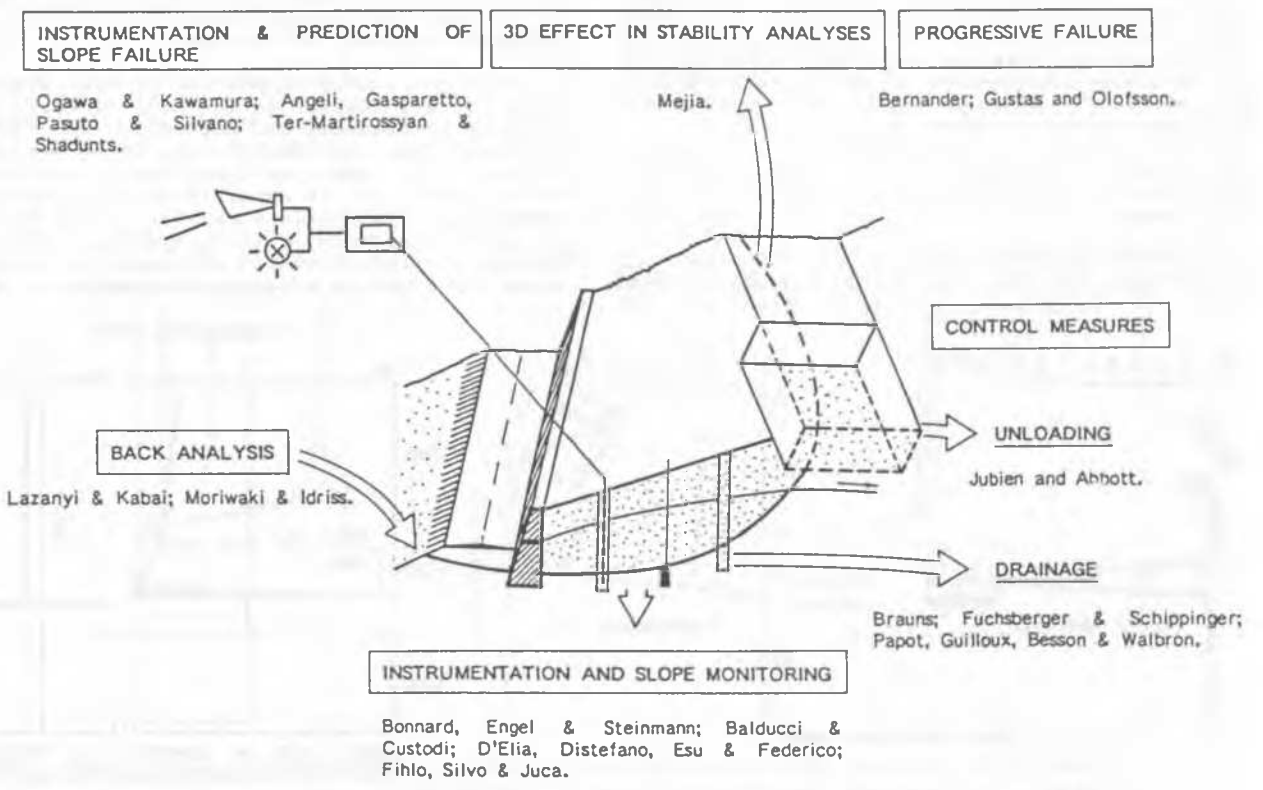


Fig 1: Thematic classification of papers presented at the Session

non-continuous movement data as well as projections beyond periods of measurement, in search of time to failure of a landslide event,may also be misleading, Bhandari (1988). For example, intermittent measurements would be inapplicable to landslides that fall in the zone of tertiary creep, virtually racing towards failure. Only for the cases in which landslide movements are very slow & take place in short spells, between large periods of no activity, that the intermittent measurements may be justified.

Continuous Measurements Through Fixed Inclinometers

Several papers in this session lay accent on continuous measurements through fixed inclinometers. A great variety of them operating on different principles and suitable for different situations, yielding varying degrees of accuracy are presented in Appendix II.

The selection of a particular type of fixed inplace inclinometer will depend on the type of slide, the purpose of measurement location, range of measurement, responsetime, accuracy, life expentancy & the cost. Some of the vexing issues, which usually defy easy decisions, are :

(1) Most critical locations so as to minimise the number of measurement points.

(2) Most probable range of measurement so as to maximise accuracy. Also, the manner in which the accuracy of the measurement is related to the range of measurement.

(3) Life expectancy of an instrument, not only from the considerations of robustness and long range reliability, but also from the point of view of economy that the instrument will not be lost to landslide because of the sheer magnitude of possible slide and degree of uncertainty associated with its behaviour.

(4) Degree of reliability of interpretation particularly when using rather Cruder brand of inclinometers.

Yet another factor of considerable importance is to deciphor from the shape of displacement depth curve if disturbance during planting of the instrument, it's very presence within the slope, relative stiffness and its interaction with surroundings were in any way responsible for modifying the actual behaviour. In the literature, one finds a great variety of depth-displacement relationships. In some cases, these relationships might yield, a clear cut judgement on depth to slip surface (Fig.2). There are cases, however, in which basal slide boundary can not be clearly defined (Fig.3). Yet another class of measurements correspond to Figure 4 which tells very little about the location of a basal slide boundary. Also, the observed movement pattern, at times, do not match the one expected on the basis of type of landslide or kind of mass movement. For instance, a **plug type of movement** is typical of mudslides in stiff, fissured clays of England and any deviation from it, if observed through measurement, will naturally raise a question mark.

Bonnard, Engel and Steinmann have added to the list a yet another device meant for measurement of continuous movements of a landslide. It marks improvement over some of the existing ones interalia,because Monotorn invar wire stands replaced by a twisted Kevlar wire, which is extremely light, easy to use as a string and is com-paratively cheaper. The device, described by Engel (1986), is also suitable for use under extreme climatic conditions besides being easy to transport, simple to maintain and accurate to measure.

On the strength of continuous measurements mode with the help of the new device at Villarbney and Loye landslides,the movement of **jerky** nature and reported & analysed to understand reasons of observed rather discontinuous behaviour. Weekly or monthly topographic observations would have failed to express the phenomena.

Reliable continuous monitoring of deep displacements of a landslide on the hill of Perugia is reported by **Balducci and Custodi.** The authors conclude that by positioning of a series of probes in the same position inside a inclinometric tube, it is possible to have continuous monitoring of displacements which could be easily made to actuate an alarm system. The displacement profile within the zone of movement could be found by appropriate moderation of the

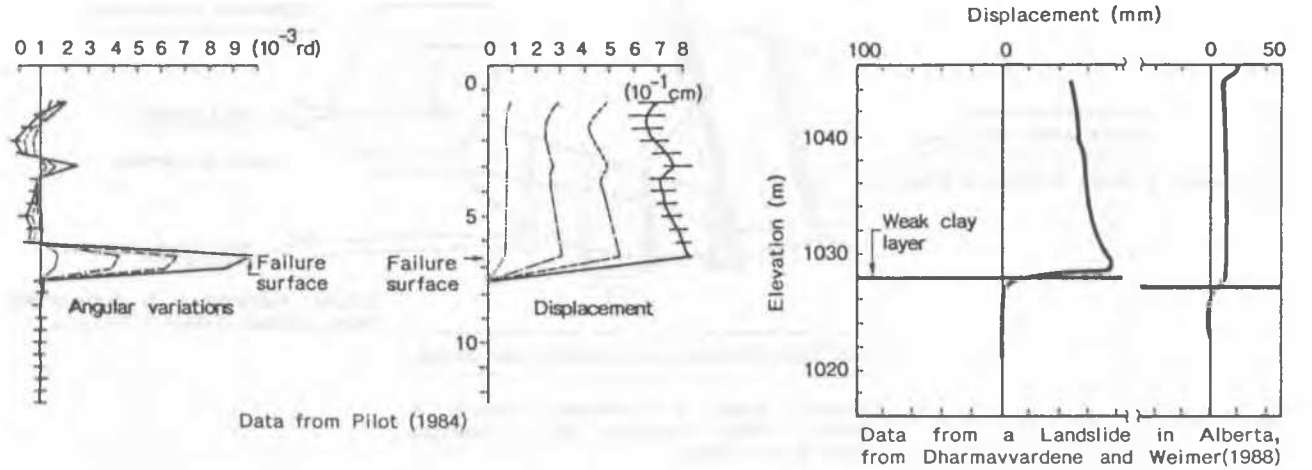


Fig. 2 Shapes of Depth Displacement Curves in which basal slide boundary could be reliably identified.

actual tube jagged curve. The paper also makes a suggestion that the number of sensor probes essential for continuous measurement could be reduced by bringing closer the sensors located away from the slip surface.

D'Elia, Distefano, Esu & Federico have presented an outstanding analysis of deformations of a 150 m - 200 m high cut in the open pit brown coal mine of St. Barbara (Arezzo, Italy). The site geology is complex, consisting of a **melange** of clay shale and rock inclusions. The authors describe a highly successful instrumentation scheme which yielded continuous data on displacements on the slope surface and in depth extended over a period of 10 years. The information on soil displacement at deeper depths was obtained by means of a series of 32 fixed in place biaxial inclinometers down to depth of about 100 m and installed at 3 m distance from one another. When displacements occurred below the maximum depth of instrumented casing following deepening of excavation, more fixed in place inclinometers are installed. The results led to improved understanding of the kinematics and mechanics of deformation processes. The paper is an extension of the earlier studies presented by the authors at ISL in Toronto (1984) & Laussane (1988).

On the strength of the inclinometric observations to depths upto 200 m, the authors explain the peculiar character of the observed deformation mechanism. They take recourse to a simplified model of a slope under conditions of active viscous-plastic flow to explain the initial deformation of the cut slope under condition of confined viscous-plastic flow and the gradual passage from conditions of confined to unconfined flow. The occurrences of "counter rotations" in the displacement profiles and of "counter scarps" on the ground surface are explained on the basis of the very same model.

A common Geotechnical Engineer will find it rather difficult to comprehend the reported mechanism and even the specialists should like to deliberate on the possible alternating explanation(s) in the backdrop of observed facts. The installations as deep as 200 m are not very common and naturally authors experiences on practical aspects of installation would be of significant value to the profession. Formation of counter scarps has been earlier discussed by Hutchinson (1988).

Filho, Silva & Juca in their paper Field Instrumentation related to Downslope movements attempt to strike a relationship between ground movements, precipitation, seasonal fluctuation of pore pressures and the width of the cracks in the buildings supported on the slopes depicting slow ground movement. The case records relate to hills of Olinda in Brazil 15 m to 60 m high with slopes varying from 6° to 45° which support many historical monuments under distress primarily due to slope movements. The surface deposits at Olinda comprise loose to poorly compacted sands and silts having variable depths upto 6 m resting on an over consolidated stiff silty clay layer with thickness varying from 4 m to 13 m. Foundations of most of the damaged monuments are laid on this stiff-silty clay formation. The measurements reported by authors include tilt of walls, opening of cracks,horizontal displacement of ground,ground water fluctuation and rainfall. Inclinometers were installed for the measurement of sub slope movements in general and for elucidation of sliding surface in particular. Widening of cracks due to building movements were monitored through extensometers with the sensitivity of about 0.01 mm. Casagrande open standpipe piezometers were used for monitoring piezometric pressures. On the strength of inclinometer measurements, the authors conclude that the movements are confined to top layer, decreasing from surface downward. The foundation settlements inside the moving layer are reported to be 4.6 m in Carmo, 2.90 m in Sao Bento and 6.0 m in Sao Francisco. The theoretical estimate of settlements of such a large magnitude is not discussed. The auhors might

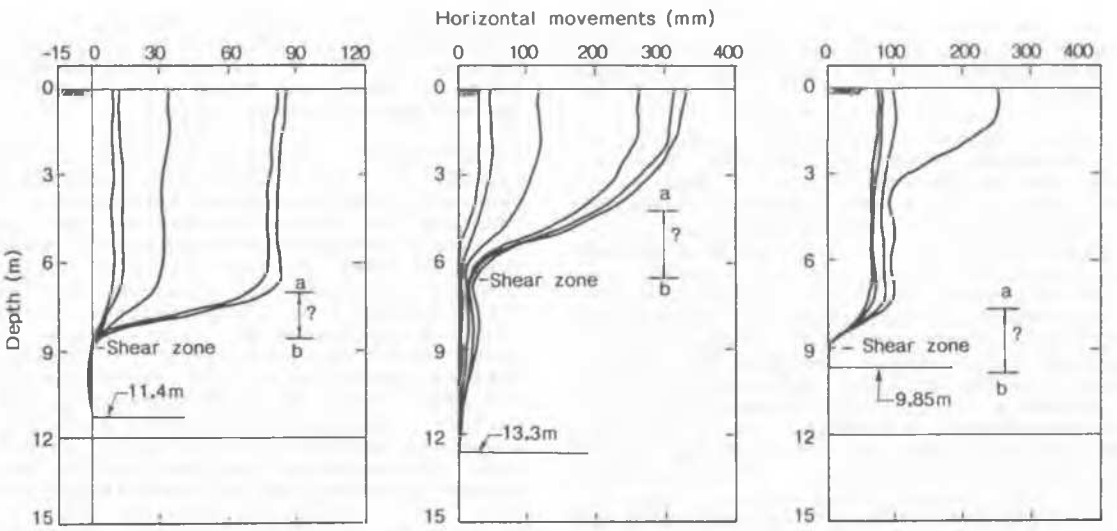


Fig. 3 Shape of Depth Displacement Curves in which basal slide boundary can not be clearly defined. (Data from Slope Instability in Coliuvial Slopes of Rio de Janeiro by Pedrosa et al. 1980).

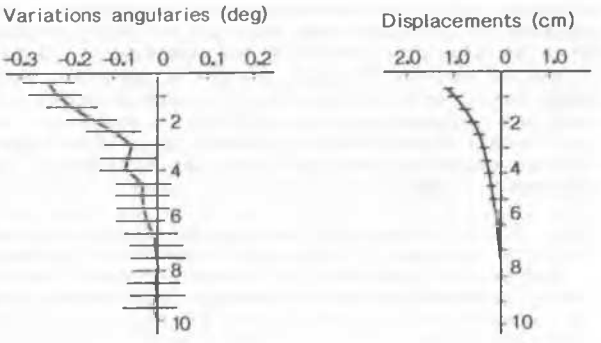


Fig. 4 Depth Displacement Profile which fail to yield any definitive Conclusion on the location of Basal Slide Boundary (Data from Cartier et al., 1981).

like to react of **negative drag** is also seen as an element responsible for settlement.

The authors present field data on rainfall, piezometric levels, widening of cracks, sub slopes displacements etc. for a period from 1983 to 1987 & conclude that instrumentation of three slopes in Olinda, Brazil, led them to establish rates of movement both during winter and summer months & relate them to hydrological conditions and fluctuations of the piezometric levels. Authors report good relationship between movements and rainfall intensity. The slope at Sao Bento site was stabilised by deep drainage and underpinning operation was undertaken at the Sao Francisco site.

LANDSLIDE PREDICTION

Ogawa and Kawamura in their paper titled "Topograhical consideration for landslide prediction" opines that the problems of prediction could be placed into the following two categories:

(1) Prediction of event of a landslide approaching failure or of a stable slope under threat of failure.

(2) Delineating potentially dangerous areas on the basis of probability of landslide occurrences in future.

Citing the case record of a typical landslide in Tertiary mudstone formation of the Central Region at the sea side of Japan in the Noto Peninsula, the authors conclude that frequent landslide occurrences are chiefly the consequences of ground water run off from the surrounding valleys. A methodology for landslide prediction is propounded based on the so called unique indices for defining topographical conditions around a slide area for evaluating characteristics of ground water runoff. Based on the actual measurement and the proposed methodology, the authors present a relationship between reduction of factor of safety (F - 1)/F and topographical conditions largely represented by drainage basin area and length of main valley. They conclude that greater the drainage basin area or shorter the length of the main valley, the more would be the reduction of safety factor. It is suggested that other indices can also be shown to influence safety factor. The authors suggest that the characteristics of ground water inflow into the landslide area can be evaluated with the total score of all these indices defined as E value. The greater the E value, the larger are the rates & amount of ground water runoff & in this way surrounding valleys do contribute dangerously to the occurrences of landslides.

The crux of the message is that it is not adequate to judge the stability or otherwise of a slope regardless of the influence of surrounding valleys. In other words, a given slope may be stable but may not always remain so if surrounded by dangerous valleys that adversely influence the slide area due to the interaction effects and drainage.

Angeli, Gasparetto, Pasuto and Silvano cite an example of landslide Instrumentation comprising an automatic recording system, utilized in slopes of different geological and climatic environments in Italy. The observational data proved significant in identification of failure mechanism. This paper describes the automatic recording system with particular reference to collapse recording of Giau Pass landslide and concludes that although the automatic recording system performed generally satisfactorily, some installation problems were faced particularly due to the poor fixing of steel wire across the slip surface down a bore hole. Pulling out of the wire consequent upon slope movement resulted in displacements at the top of the bore hole many times higher than those obtained by means of steel wire connected to the automatic device. In fact, the ratio between the displacements obtained by topographic measurements and those by the automatic recorder ranged from 5 to 150. The remedy to slippage of bore was found by deepening the anchorage to well below the slide boundary as well as by improving the anchorage in itself. Further improvement was effected by locating measurement points such that anticipated movement on those were either entirely vertical or entirely horizontal. This became necessary because the device was unfit to yield vertical and horizontal components of movement separately.

Following the inverse of velocity versus time relationship as proposed by Fukuzono (1985), forecasting of landslide was attempted and a fair agreement was found between the actual and the estimated time of failure.

There are a few case records of successful production. Saito (1977) claimed to have predicted the Takabayama landslide of January 1970 well in advance of failure time and reports a close correspondence between the predicted and the observed failure time. Voight (1988) reports validity of approach in predicting failure time of St. Helens volcanic eruption and of the rockslide of Mt. Toc - Italy. He states that "with the inverse rate, a prediction of failure date could have been made in late September, more than 10 days before the actual failure. The high velocity of the slide created a surge of water in a reservoir, over topping a dam, killing more than 2000 people, and causing losses totalling 200 million (1964) US Dollars".

The prediction of slope failure of 18 Feb., 1969 in the Chuquicamata mine, Chile, five weeks in advance of it's occurrence is yet another example documented in detail by Kennedy et al.

(1971),Kennedy & Niermeyer (1979) and Voight and Kennedy (1979). Fukuzono (1985) has also claimed success of the approach for experimental slope failures. The reasonableness or otherwise of such predictions are discussed by Bhandari (1988).

Ter-Martirossyan & Shadunts in their paper deal with scientific prognostication of the landslide processes on construction sites. They tend to link the landslide behaviour with the geological characteristics of cohesive clay slopes and suggest that during the life time of a building on slope, the stress strain distribution within the slope can be estimated from the knowledge of movement velocities.

The authors opine that quantitative forecasting of landslide dynamics should include all stages of development. In other words, it must embrace the initial, the intermediate and the catastrophic phases of development. The initial phase represents the initial Strain Stressed State (SSS) of the slope and may be attributed to factors like construction, change of slope profile and change of hydrorheological conditions. The authors state that, in this phase, the landslide deformations develop intensively but eventually stabilise to enter into the second stage. The second stage refers to the relatively stabilised SSS of the slope in which deformations, either at constant or variable rate, are within permissible limits during the life of the project. The third stage refers to landslide acquiring a progressively increasing rate of displacement bringing about catastrophic slides. All this ends up with surface sliding of large volume of soil or rock terminating with newly found state of equilibrium characterised by the cycle of SSS.

The authors enunciation as outlined above, seem to fall in line with the classical concepts of primary, secondary & tertiary creep well studied in geotechnical engineering (Ter-Martirossyan & Shadunts). The authors present the case of forecasting of landslide process dynamics of slopes formed with water saturated clay soil and non-water saturated loessial soil.

Progressive Failure

Most stability analyses either disregard the mechanism of progressive failure or tend to account for it in a highly simplified fashion. **Bernander, Gustas, & Olofsson** present an improved model for progressive failure analysis of slope stability. Bernander et al. (1978-84) had earlier shown that a minor disturbance to a slope of strain softening material may trigger a failure which may eventually propagate into vast areas even in flat terrains. The authors have developed the progressive failure model further by discretising soil mass not only in the down slope x-direction but also in the perpendicular z-direction, thereby permitting modelling of deformation in the failure zone. The authors consider the constitutive relationship in respect of the failure zone in two stages to provide closer approximation to conditions prior to formation of slip surface and those after failure. The time dependency of the modulus of elasticity as well as external arbitrary vertical or downslope loads can be accommodated in the progressive failure analysis for which a computer programme is now available. The authors claim usefulness of the proposed model for evaluating not only the insitu stress but also the additional effects of superimposed loading. The merit of the approach lies in recognising the factor of time in analysis.

It is well established that limit equilibrium approach assumes **failure** simultaneosly on the boundary of shear. Thus factor of safety is regarded to be constant and **progressive effects** get ignored in the process. In order to account for progressive failure one needs to, interalia, know initial state of stress, time dependent distribution of incremental stresses on the boundary of shear and ensuring failure pattern. In a recent work, Bertoldi (1988) considers two different approaches to develop method of progressive failure analysis. These are :

(1) to consider initial stress field in the identification of local failure & then to follow up with same stress redistribution mechanism;

(2) to simulate progressive failure as a transformation from initial stress field to the stress field associated with limit equilibrium.

In such a case, two different types of analysis are required for two parts of the sloping mass at each stage of progressive failure. As the failure progresses, the relative size of these masses change. The interaction between the two masses is taken into consideration by including limiting earth pressures as extreme cases of possible interaction.

It is also now well established that influence of progressive failure on stability is influenced not only by soil shear strength & brittleness factor but also by slope and slip surface geometry. Mathematical modelling of the phenomena would, therefore, remain a very feasible area for research for many more years to come.

BACK ANALYSIS OF FAILED SLOPES

Back analysis is an extremely powerful tool generally used to derive the maximum advantage from the investigation of a failed slope. It aims to estimate shear strength parameters essential for the design of remedial measures. The cases one normally comes across are :

(a) Back analysis of the steepest & the tallest stable slope in a given geological setting with symptoms of creep, instability or failure, in order to work out nearly the upper bound of shear strength;

(b) Back analysis of failed slopes & landslides soon after their occurrences, availing of the fact that factor of safety is unity at the state of limit equilibrium. Considerations of mechanism, history,slide boundaries,materials characterics, geological details, pore pressure conditions etc. of a landslide would, of course, require high quality statistically reliable data;

(c) Back analysis of failed slopes or landslides with the worst assumption of pore water pressures (which are not often reliably known) in order to estimate the lower bound of shear strength.

The basic connected issues of high importance are :

(a) Whether the back analysis is to be performed in terms of total stress or in terms of effective stress ?

(b) How reliably the slide boundaries could be delineated in fullest consideration of inter-alia slope topography before and after failure, stratigraphy, hydrology and geology including minor geological details ?

(c) How the degree of uncertainties associated with decision on assumptions of input parameters be minimised through parametric study ?

(d) Whether or not the consideration of 3D effect is significant in a given situation and if it is, then what is the methodology to be followed to make the stability analysis ?

(e) Can the insitu stress and strain history of the slope,the progressive failure & time effects be adequately reflected in a back analysis ? And if it cannot be, to what implications ? Does the rate of slope failure become significant at any time ?

(f) How the special situations involving complex geology, water filled tension cracks, multi-tier sliding, artesian pressures, earthquake effects etc. be handled in a real situation ?

It is generally accepted that the back analysis of a nearly homogenous saturated slope is best done in terms of total stress. On the other hand, when dealing with situations in which reliable information on pore pressure is available, an effective stress analysis is always preferable. It is customary, in such cases, to follow the procedure outlined below:

(a) Assume the most probable value of angle of shearing resistance in order to estimate cohesion intercept or vice versa.

(b) Analyse main cross section of a failed slope and another section close to the main cross section in the same failed slope so to establish two equations, from which values of c' and Ø' can be evaluated.

(c) Utilize one cross section each of two different failed slopes which are reasonably identical from geotechnical point of view in order to establish two equations for evaluating the two unknowns c' and Ø'.

Numerous investigators have critically analysed the potential, procedures & limitations of back analysis, He (1988), Sauer and Fredlund (1988), Fredlund,Crahn & Pufahl (1981), Papadopoulos and Anagnostopoulos (1981),Tinoco (1981), Widdis and Clapham (1981),Fontoura,Decampos & Filho (1984), Hencher, Massey and Brand (1984), Leroueil and Tavernas (1981). Many investigators have reported excellent correspondence between the shear strength parameters estimated through back analysis and those measured in the field and the laboratory investigations. There are many others who find severe limitations largely due to inadequacy of reliable data but so far no one seem to have made any assault on the basic philosophy and the utility of back analysis.

Two of the papers presented for discussion at the session take recourse to back analysis of failed slopes.

Lazanyi and Kabai presents an interesting Case Record of Stability of Spoil Heaps Dumped rearwards of an abandoned excavation in the Visonta Lignite mine in northern Hungary. Heaps comprised grossly complex mixture of volcanic soils of high plasticity, poorly draining soils of low or little plasticity and freely drained soils. The several individual layers in the mine were excavated using bucket wheel excavator and hauled through nearly a kilometer long conveyor belt from where the placement was achieved by dropping from a height in layers of 20 m using spreader machines. Wide berms were provided to punctuate the slope with successive steps of stability. The authors report occurrence of mudflow, where the placement was at high water contents; slump failures where drainage was rather inadequate; & failure due to build up of excess pore pressures in situations where dumping was rapid. Some slides involved nearly one million cu.m. of spoil. Adopting the back analysis approach and based on stability considerations, tentative charts were developed for the determination of terraced spoil heap profile.

A highly significant point made in the paper is the recognition of the fact that excess pore pressures would develop as a consequence of loading due to dumping. The evidence of loading induced high pore pressures developed in mud slides in the London clay of the Isle of Sheppey was earlier provided by Hutchinson and Bhandari (1971). One of the lessons from the reported study is, therefore, that one must consider various ways in which pore pressure gets mobilized at the slide boundaries in order to arrive at a more realistic estimate of the shear strength at failure by Back Analysis.

The paper by **Moriwaki, Idriss, Moses and Ladd** presents reevaluation of the L Street slide in downtown Anchorage triggered due to the Alaskan Earthquake of 27th March, 1964 (ground motion = 0.15 g to 0.2 g, surface wave magnitude = 8.5 and moment magnitude = 9.2). The sub-slope profile was reliably established as a very dense gravel and sand deposit resting over alternate layers of clayey silty sands and sandy silts to depths of about 16m in the flat area. The underlying strata was identified as lightly over consolidated Cove clay to depths of about 45 m below which lay the glacial till. The possibility of failure due to liquifaction in the interbedded zone was ruled out and the probability of failure occurring in lightly over consolidated Cove clay was established by scientific reasoning. The failure reportedly occurred under undrained conditions in the upper part of the normally consolidated to lightly over consolidated soil. It's cause was not so much the loss of undrained shear strength due to earthquake trigger but rather significant loss of undrained strength due to large orders of deformations.

Analysis was done to find displacements for a selected block along a chosen section using the improved Newmark method. The undrained shear strength which would allow the observed seismically induced displacements were computed and these compared with those estimated from the site subsurface conditions in 1964. The changes

in undrained shear strength due to cyclic loading were investigated by conducting DSS followed by post cyclic static tests. Even when very high excess pore pressure ratios were induced in the specimen due to cyclic loading, the post cyclic undrained shear strength was found to be a reasonably large fraction of the precyclic peak undrained shear strength and the decrease in the strength is around 30%.

An agreement is seen between the back calculated and the observed undrained shear strengths. This paper further reiterates the need for systematic studies of case records.

There are several other papers presented to this Conference falling into the domain of Technical Session 21 which deal with **Back Analysis**, for example, **Brackley et al.; Wolle et al.; Lambe et al. and Hovland.** There is an urgent need to synthesise the information towards generic conclusions.

THREE DIMENSIONAL (3D) EFFECTS IN STABILITY ANALYSIS

Very often erroneous forecasting of instability is made by ignoring the 3D effects provided by landslide boundaries. It is a common professional practice to ignore 3D effects by taking recourse to a simpler and conservative 2D stability analysis, chiefly in view of uncertainties and pitfalls usually involved in design assumptions. There are many geological situations, however, in which such over-conservative assessment of stability is not warranted and high costs on this account cannot be justified. The relevance of 3D stability analyses assumes significance in such cases.

Mejia has presented a case record of an old earth dam for which the factor of safety against instability obtained by a 3D analysis was about 12 per cent greater than that computed by using a 2D approach. The paper points out that the ratio of length of the slope to its height (L/H) represents a highly significant factor. The results presented by Mejia reproduced in Table 1 clearly indicate that for the downstream earthquake situation in the case considered by him, the 3D effect is highly significant. For homogeneous cohesive soils, with L/H ratio varying between 2 and 6, the improvement of factor of safety seem to lie in the range 10 per cent to 30 per cent, which may not be regarded as significant, particularly if input data are not so reliable.

Table 1 : **Comparison between 2D & 3D factors of safety**

Loading conditions	Factor of Safety		Difference (%)
	2D	3D	
Downstream long term	1.46	1.56	7
Downstream earthquake	0.58	1.25	116
Upstream rapid drawdown	0.96	1.05	9
Upstream pseudo-static (K = 0.15)	1.05	1.15	10

Studies on the 3D effect of slopes have been made among others by Sherard et al. (1963), Anagnosti (1968), Baligh and Azzouz (1975), Chen and Chameau (1982) and Dennhardt and Forster (1985). In fact, Kennedy (1956), on the basis of his study of landslides with cylindrical slip surfaces, concluded that side effects seldom involve errors above 6 per cent. Based on the studies on a mudslide at Beltinge and at the Bouldnor Cliff in UK, Bhandari (1971) had demonstrated that errors in the case of mudslides lie in the range of 12 per cent to 18 per cent.

CONTROL OF LANDSLIDES

One of the prime objective of the studies on multifaceted aspects of slope and its behaviour is to evolve cost effective control measures ensuring short and long term safety. Ways and means of controlling a landslide is a subject that involves very vast scope & scale of operation. The two contributions to this session deal with landslide control (a) by drainage and (b) by unloading the problematic slope by cutting at its head.

Brauns lays accent on planning and regulation aspects of drainage systems in order to effectively deal with mechanical, hydraulic, geological and chemical influences which often tell upon the efficacy of vertical and horizontal drains provided as measures of landslide control. Most of the current practices preclude the possibility of inspection and checking of drains thereby introducing uncertainties about their long range satisfactory behaviour. Brauns, while critically analysing the reasons for ageing of drainage systems, advocates the need for planning and regulation of drainage systems. He points to the essentiality of provision of flushing in the event of impaired efficiency of filters.

The two examples cited in the paper vividly illustrate the above points.

Fuchsberger and Schippinger advocate the use of drainage screen to achieve stabilisation effect for the case of concrete bridge on a sliding slope. Instability of the two foundation piers due to creep down hill, was attributed to high pore pressures obtained in the sandy seam below. The provision of drainage screen was intended to relieve the sandy seam of excess pore pressures thereby improving the effective stress and, therefore, the resistance to shearing. Taken conjointly with the under-pinning, the stability could be restored as testified by the field observations on piezometers, settlement gauges and inclinometers extended over a period of several years.

The R&D on drainage systems, including development of tools and techniques, appropriate to a variety of geotechnical situations have undergone a major transformation during the past two decades. Wherever the question of surface or near surface drainage assumes importance, contour drains, catch water drains, trench drains etc. normally suffice. The real difficulties arise in dealing with major landslide problems requir-

ing deep drainage. The spatial distribution of drains, their type, design, installation technology, short term and long term efficacy and the overall benefit that accrues to the landslide are all matters not emanable to a simple, single answer. Since very high costs are normally involved in the treatment by deep drainage, it has been customary to be rather conservative in design than to face recurring problems of instability & maintenance by underplaying in design. Of the various advanced technologies available for deep drainage, the one developed & patented by Rodio deserves special mention. The technique essentially aims to create a curtain of large vertical drains interconnected for efficiency of drainage by gravity, Sembenelli (1988); Collata, Manassero and Moretti (1988).

Despite the volume of worldwide experience on control of landslides by ever improving techniques of drainage, the difficulties of linking the requirements of drainage, methodology adopted, benefits realised and eventual short and long term implications continue to exist. Synthesis of more and more case histories alone could help arrive at generic conclusions towards improvement of drainage technologies and also forecasting methods to arrive at a realistic assessment of the efficacy of a drainage system. Instrumentation and field monitoring of pore pressures deserve to be treated as an integral part of any recommended scheme of drainage for landslide control.

Papot, Guilloux, Besson and Walbron present an interesting case record of landslide control by drainage together with anchored diaphragm wall. The slope which supported a distribution substation was affected by slow but regular movements for the past several years. The slope was generally mild (10° to 20°), it was made of plastic clay colluvium (I_p = 25 to 30) with some gravelly sand lenses for about 20 m thickness and water table coincident with the ground surface. The observations of displacement spread over a period of 38 months revealed that the movements were on the order of 1 to 2 cm per year. The triaxial test revealed residual angle of shear resistance of 10° and cohesion intercept of zero.

Of the various available options, deep drainage was selected as the solution with 10 m to 15 m deep walls connected with 30 to 50 m long sub horizontal radial drains were at 2 or 3 levels and about 10 drains provided to at a level. An anchored wall was provided for added strengthening.

The authors claim that lowering of groundwater table by provision of deep drains contributed 10 to 18% improvement in safety factor. The diaphragm wall contributed another 10 to 15%. And the two together boosted the factor of safety by 27 to 36%. Since the work has only recently been completed, the long range efficacy of the system will have to be assessed based on observations yet to come.

Interestingly, there are cases of problematic slopes in which investment on drainage may prove infructuous. If slopes are already well drained, they may not require any such measure. This is clear from the case record of a landslide stabilised by unloading by cutting at the head of a slope reported by **Jubien and Abbott** in their paper to the session. The case refers to an unstable hill side on the northern shore of Kamloop lake on the Canadian national Railway main line. Although the slope had a history of erratic, infrequent movements, the rate of movement gradually increased after 1985. When drainage was considered as a remedy for the problem, calculations revealed that the improvement in stability by provision of horizontal drains would be scarcely 5 per cent. Even if the piezometric surface was to be lowered to an elevation of 20 m, the increase in stability was expected to be only about 17 per cent. Obviously drainage was dismissed as a remedy in this case. On the other hand, simple unloading operation through excavation of some 60000 cu.m. of the materials from the head of the slide was estimated to decrease the driving force to about 60 per cent of it's original value. The ultimate recommendations involved acceptance of a certain degree of risk by achieving only partial stress relief through unloading. The measurements taken after unloading have vindicated the effectiveness of the treatment.

Low Cost Landslide Control Measures - New Ideas

Cost reduction in treatment and control of landslide is one of the greatest challenges of our times particularly for countries like India in which such phenomena are very wide spread. Of the various control measures, reinforced cement concrete and masonry retaining walls are very common. They are very expensive, however, and do call for import of cement, steel, stones, sand and water from long distances on the hills. In order to combat the situation, a novel low cost anchored drum retaining wall technology was developed at the Central Building Research Institute in India. It makes use of slope waste itself thereby eliminating the need for transportation of construction materials & effecting saving to the tune of 40 per cent in cost, Bhandari (1987, 1988).
Slope waste, colluvium, landslide debris, tubes accumulation etc. usually contribute extensively to development of debris flow on slopes. Every

Fig. 5 A View of Landslide at Kaliasaur in India where An Anchored Drum Retaining Wall was constructed.

Fig. 6 Anchored Drum Diaphragm Retaining Wall at the Kaliasaur Landslide in India.(Bhandari, 1987).

major landslide in India, for example, generates nearly 5000 tonnes of debris annually. Even in stable natural slopes, every km of road could be expected to generate 2000 tonnes of debris right at the time of construction. The proposed retaining wall construction technology deploys the debris gainfully. One such wall constructed at Kaliasaur in Garhwal Himalaya which represent a major landslide is shown in Figure 5 and Figure 6. The techniques,as described in earlier pages,makes use of empty bitumen drum containers which are packed with landslide debris to obtain gravitational effect and are inter-connected appropriately to obtain an anchored bulkhead effect.

The design methodology for the drum diaphragm wall was earlier discussed and the wall actually constructed at Kaliasaur has already stood the test of time as no serious problem has been encountered at the site for the last three monsoon seasons, Bhandari (1988).

The technology has now been further extended to provide several options. The matrix material in colluvium which is generally cohesionless, could be used in the construction of reinforced earth walls. The coarser fraction including stones and boulders could be used in the construction of gabion walls. The recent work at CBRI indicates strong possibilities of using (a) ferrocement technology; (b) reinforced brick panel system and (c) stone masonry blocks to replace empty bitumen drums. Wire netted sausage walls are currently extensively being used all along the Himalaya as also the netlon or tensar baskets,which are prohibitively expensive in Indian conditions.

Three technologies are being offered by CBRI in ferrocement construction. The first deploys ferrocement cylinders (Fig.7), the second promotes the use of ferrocement rectangular wells (Fig.8), and the third make use of ferrocement triangular units (Fig.9).

In areas where bricks are readily available, the container module can also be made of reinforced brick panel system developed at the CBRI (Fig.10). Yet another possibility is to manufacture stone masonry blocks by making use of stones and rock rejects. Stone blocks are then used to create containers (Fig.11). The technique of making stone blocks is described in Appendix I.

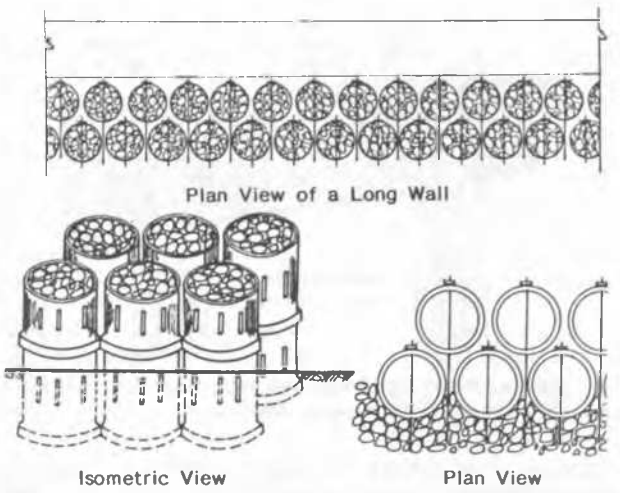


Fig. 7 Landslide Debris filled Ferrocement Cylindrical Units developed at CBRI in India.

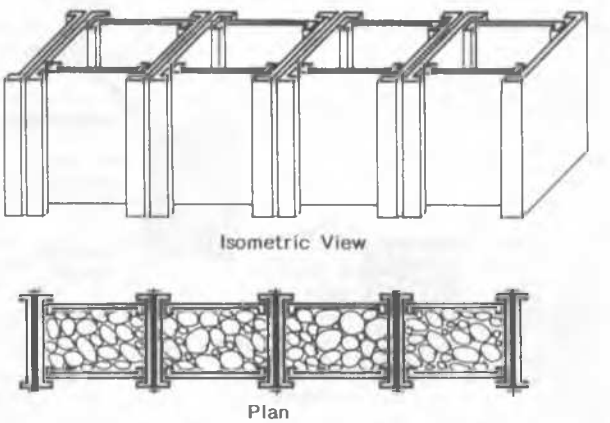
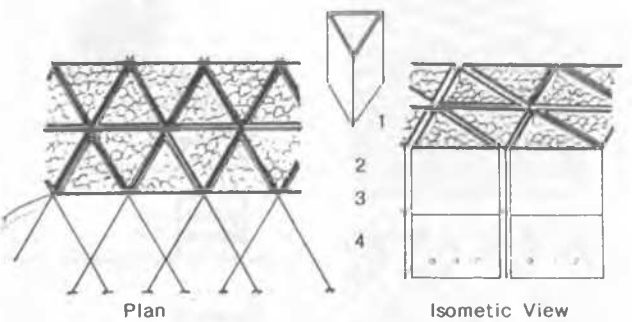


Fig. 8 Landslide Debris filled Ferrocement Rectangular Units for Retaining Wall Construction developed at CBRI, India.



(1) Landslide Debris; (2) Triangular Units; (3) Weep Holes; (4) Anchors.

Fig. 9 Landslide Debris filled Ferrocement Triangular Units for Retaining Wall Construction developed at CBRI, India.

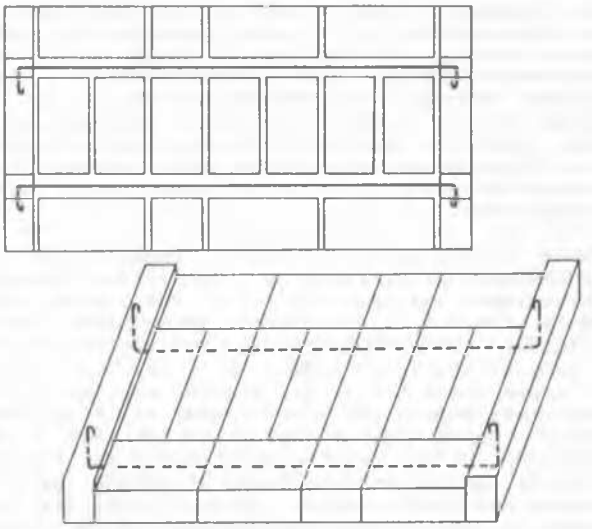


Fig. 10 Prefab Brick Panel System - developed at CBRI, India.

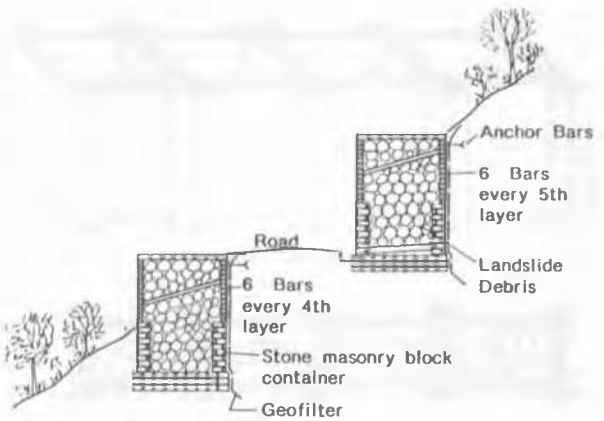


Fig. 11 Landslide Debris-filled Stone Masonry Wall Container developed at CBRI, India.

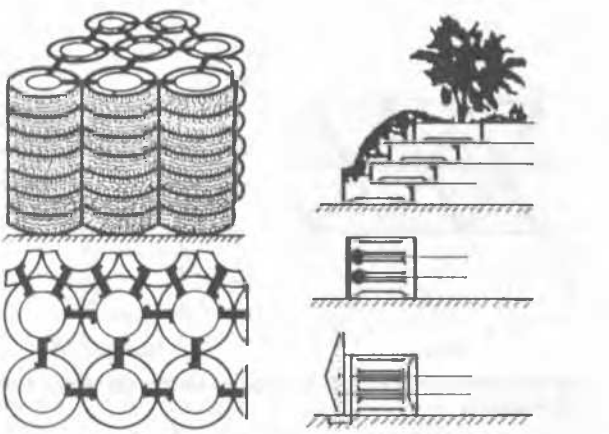


Fig. 12 Retaining Wall using Used Tyres to support Urban Refuse (Cartier, Long, Pouget, Bargillat and Cudennec, 1981).

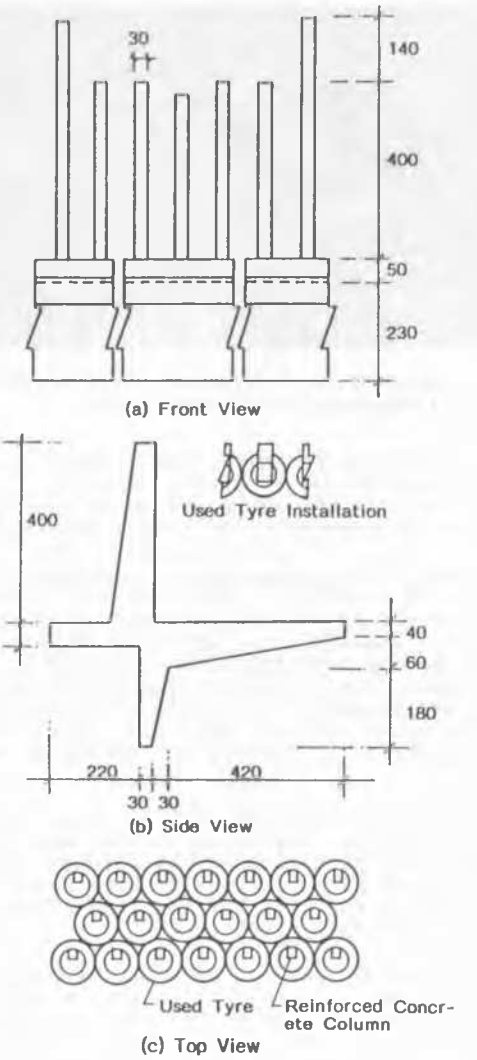


Fig. 13 Utilisation of Used-Tyres as Cantilever Check Dam (Tuan, 1988).

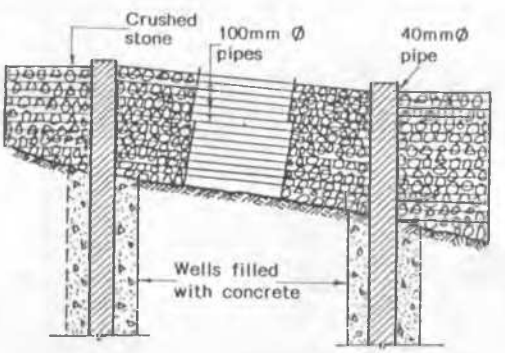


Fig. 14 Retaining Structure with Crushed Stones (Behpoor and Ghahramani, 1988).

Cartier et al.(1981) & Tuan (1988) have described techniques involving utilisation of **Used Tyres** in creating respectively a retaining wall and a check dam (Fig.12 and Fig.13). Behpoor

and Ghahramani (1988) have suggested a retaining structure using crushed stones (Fig. 14).

All of the above techniques do hold considerable promise as alternatives to the costlier reinforced concrete and stone masonry walls.

CONCLUDING REMARKS

Some of the worthwhile topics of discussion emerging out of the papers presented to the Session can be summarised as follows :

(1) Fixed-in-place inclinometers for continuous monitoring of slope displacements versus periodic measurements with particular reference to (a) type of landslide and other mass movements and their anticipated behaviour (b) landslide prognostics.

(2) Various models of progressive failure - philosophical issues analysis of slopes.

(3) Relevance of three dimensional stability analyses in the face of inherent pitfalls and complexities usually associated with study of slopes.

(4) Power, potential and limitations of Back Analysis.

(5) Design considerations and long range efficacy of Deep Drains.

(6) Low Cost Control Measures and their effectiveness.

References

Behpoor, L., Ghahramani, A. (1988), Landslide and Stabilization at Gas Pipeline Station 201 in Iran, Int. Symp. on Landslides, Lausanne, Vol.2.

Bernander, S. and Gustas, H. (1984), Consideration of Insitu Stresses in Clay Slopes with Special Reference to Progressive Failure Analysis, Int. Symp. on Landslides, Toronto, Vol.1.

Bhandari, R.K. (1970), Mudflows in Stiff Fissured Clays, Ph.D. Thesis, University of London (Unpublished).

Bhandari, R.K. (1980), Prediction of Landslide Behaviour, Instrumentation etc., General Report, Proc. Int. Symp. on Landslides, New Delhi, Vol.3.

Bhandari, R.K. (1984), State-of-the-Art Report on "Simple and Economical Instrumentation and Warning Systems for Landslides and other Mass Movements", Proc. of the Int. Symp. on Landslides, Toronto, Canada, Vol.1.

Bhandari, R.K. (1987), Slope Instability in the Fragile Himalaya and Strategy for Development, IXth Indian Geotechnical Society Annual Lecture, Indian Geotechnical Journal, Vol.1.

Bhandari, R.K. (1988), Some Practical Lessons in the Investigations and Field Monitoring of Landslides, Vth Int. Symp. on Landslides, Lausanne, Vol.1.

Cartier, G., et al. (1981), Railroad Slope Instabilities: Monitoring and Prevention, X ICSMFE, Stockholm, Vol.3.

Colotta, T., et al. (1988), An Advanced Technology in Deep Drainage of Slopes, Int. Symp. on Landslides, Lausanne, Vol.2.

Fukuzono, T. (1985), Method for Predicting the Failure Time of a Slope, Proc. IVth Int. Conf. and Field Workshop on Landslides, Tokyo.

Hutchinson, J.N. (1988), General Report: Morphological and Geotechnical Parameters of Landslides in relation to Geology and Hydrogeology, Proc. Vth Int. Symp. on Landslides, Lausanne, Vol.1.

Kenney, T.C. (1956), An Examination of the Methods of Calculating the Stability of Slopes, M.Sc. Thesis (Unpublished).

Kennedy, B.A., et. al. (1971), A Case Study of Slope Stability at the Chuquicamata Mine, Chile, Trans. Soc. Min. Eng., Am. Instt. Min. Metall Eng., 250.

Kennedy, B.A., Niermeyer, K.E. (1971), Slope Monitoring System used in the Prediction of a Major Slope Failure at the Chuquicamata Mine,. Chile, Proc. Symp. on Planning Open Pit Mines, Johannesburg, Balkama, Amsterdam.

Pilot, G. (1984), Instrumentation and Warning Systems for Research and Complex Slope Stability Problems, IVth Int. Symp. on Landslides, Toronto, Vol.1.

Saito, M. (1977), Evidential Study on Forecasting Occurrence of Slope Failure, Trans. of the Deptt. of Geomech. American Academy of Sciences, Yeravan, USSR.

Sembenelli, P. (1988), General Report: Stabilization and Drainage, Int. Symp. on Landslides, Lausanne, Vol.2.

Tuan, C.H. (1988), Study on the Gully Control by Used-tire Structure in Northern Taiwan, Int. Symp. on Landslides, Lausanne, Vol.2.

Voight, B. (1988), A method for Prediction of Volcanic Eruption, Nature, Vol.32.

References have not been given for the Papers presented in this Session.

ACKNOWLEDGEMENT

The assistance rendered by Shri Ajay Paul, Shri Shantanu Sarkar, Dr. Kusum Shanker, Shri Sudhir Sharma & Miss Sunayna Kohli is gratefully acknowledged. Thanks are also due to Shri Surendra Giri, Shri Khushpendra Arora & Shri N.K. Taneja for so expeditiously and painstakingly handling the text. Special thanks are due to Shri A.K. Jethi for his highly specialised role in making high quality photographic slides for the General Report.

APPENDIX 1

Utilisation of Slope Wastes in Construction of Retaining Walls

Most of the Himalayan landslides are involved in a colossal amount of mass wasting. As a consequence, thick carpets of colluvium are seen to cover most of the problematic slopes. The slope wastes and talus accumulations which are often involved in debris flows and mudflows could be looked upon as a resource material in the construction of retaining walls.

Samples of colluvium and talus accumulations were analysed from more than two dozen landslides all over the Himalayan belt. The slope cover was found to consist of ill-sorted rock fragments and boulders set in the metrix of cohesionless materials. The production of boulders to metrix was seen to vary from situation to situation.

The proposed technology envisages (a) utilisation of cohesionless matrix material in rein-

forced earth wall construction and (b) boulders and rock fragments in the construction of masonry walls, by conversion of slope rejects into regular blocks of size 30 x 20 x 15 cm.

Special moulds have been designed for battery casting of such blocks using rock waste as the core material and 1:5:8 cement concrete as the filler. Such blocks have a compressive strength on the order of 7 N/mm^2.

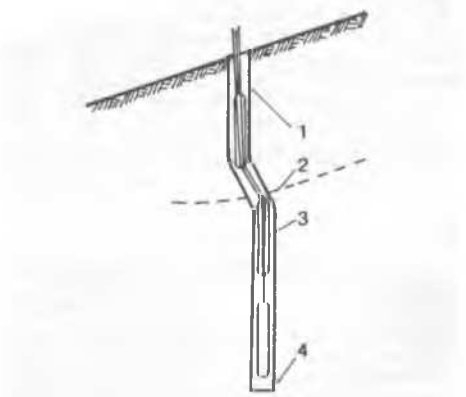
The stone masonry blocks as cast above could also be provided with anchored rods and dowels according to the design needs.

Appendix II: Fixed in Place Inclinometers.

Deflection Tube Torpedo (CBRI, India, Bhandari, 1984, Fig A1)

The system comprises a train of torpedos, each connected with a nylon string which passes through all the other torpedos above it. The torpedo train is placed in a PVC tube and the tube is installed in a bore hole.

The PVC pipe deflects due to the subsurface movements thereby preventing the free passage of the torpedo through the shear zone. The shear zone is detected by pulling one string at a time, starting from the top most torpedo, in search of that torpedo which gets stuck in the displacement zone. This instrument permits only crude estimation of depth of sliding surface but is widely used because of low cost, ease of installation and simplicity of measurement.



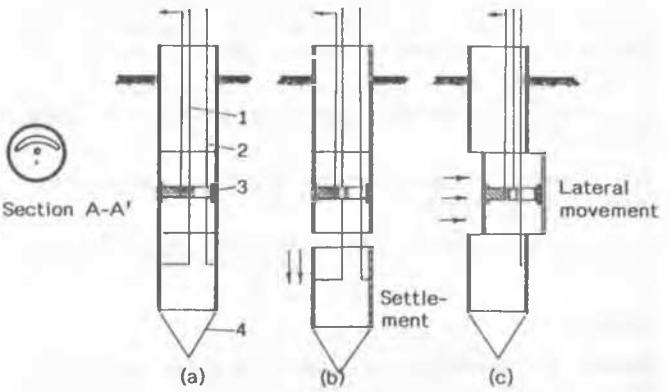
(1) Protection Tube; (2) Sliding Surface; (3) Torpedo; (4) Borehole.

Fig. A1 Deflection Tube Inclinometer.

Ground Deflection Meter (CBRI, Roorkee Design, Fig A2)

The system comprises steel pipes placed one above the other, free to slide laterally (perpendicular to the axis of the pipe). The alternating sections of the pipes have a concentric revolving disc mounted on ball bearings. The disc has a central hole and a profile cam slot. On to the disc, a rigid rod is welded which reaches the top of the bore hole and acts as a pointer for lateral movements. Two wires fixed to the bottom steel pipe section pass through the disc, one from the central hole and the other from the profile cam slot. The wires are connected to the readout unit on the top which comprises a set of dial guages, pulleys and dead weight. The wires are kept in constant tension by dead weight.

When the movement takes place, for examples at the shear zone, the pipe section at the slide zone slides laterally in the direction of the displacement thereby causing the two wires to stretch. The wire passing through the central hole provides the necessary extension to monitor the magnitude of lateral displacement. The wire passing through the profile cam slot forces the revolving disc to revolve in the direction of lateral displacement and reading corresponding to this reflects the vertical settlement.



No tension in either of the wire at installing stage

Tension increases in both the wires due to settlement

Tension increases in central wire as a result of the lateral movement

(1) Central wire
(2) Peripheral wire
(3) Slotted plate
(4) Cone

FIG. A2 GROUND DEFLECTION METER (CBRI,INDIA DESIGN)

Fixed Type Chain Deflectometer (Chain Deflectometer Manufactured by Interfels, Fig A3)

Comprises steel rods having ball and socket joints. The rods are free to move in any direction perpendicular or oblique to the axis of the borehole. On account of displacement at the shear zone, the rod section in it's proximity tends to move with the Slidemass thereby providing thrust on the deflectometer, which can be measured either by electrical or by mechanical means.

Fixed Type Multiple Wire Bore Hole Movement Indicator (Manufactured by Interfels, Fig A4)

The instrument is similar to the multiple wire inclinometer but instead of steel tubes and sleeves a number of wires are attached to flat sections. These sections are initially installed perpendicular to the bore hole axis protected by a steel tube or PVC tube casing in which the

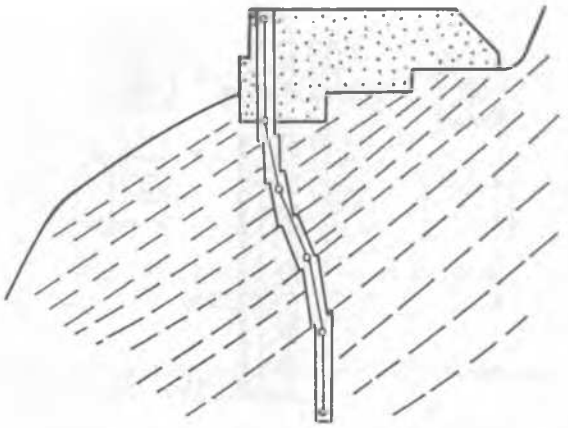
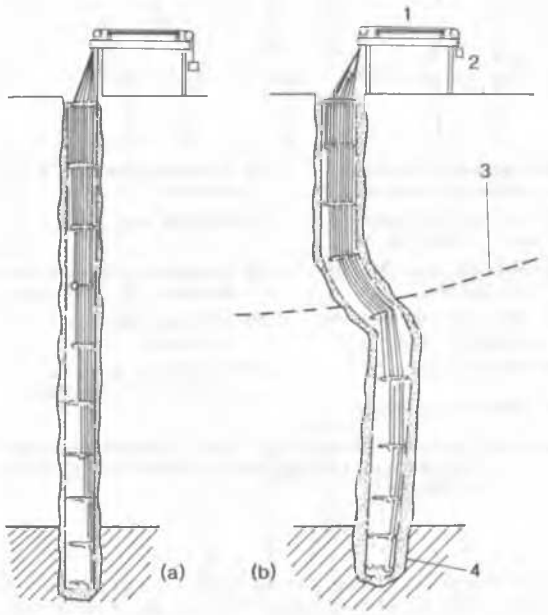


FIG. A3 FIXED TYPE CHAIN DEFLECTOMER (CHAIN DEFLECTOMETER MANUFACTURED BY INTERFELS).



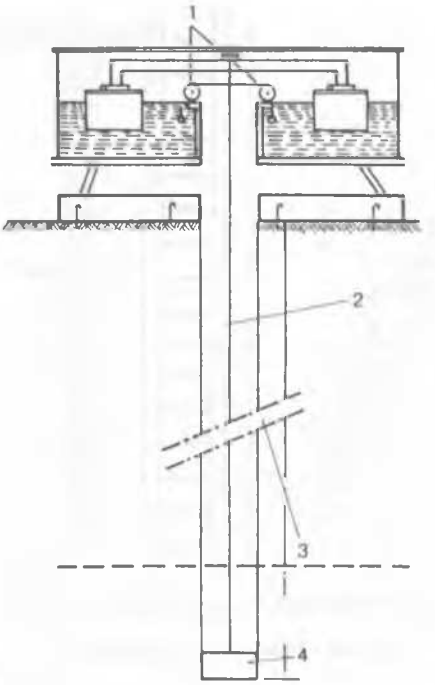
(1) Measuring Unit; (2) Dead Weight; (3) Sliding Mass; (4) Fixed Bottom; (a) Before Movement; (b) After Movement.

Fig. A4 Multiple Wire Borehole Movement Indicator.

entire assembly is placed. As the displacement takes place, the wires get stretched. The stretching of the wire is monitored at the top of the bore hole. The wires are kept constantly in tension using dead weights.

Inverted Pendulum Type Inclinometer (Delmas and Rodriguez, 1984, Fig A5)

A length of wire secured at the bottom of the bore hole as well as on the top serves as an inverted pendulum. Two wires are attached at the top end of the stretched wire, which in turn are coupled to two potentiometers. When the deflection takes place, the stretched wire tends



(1) Potentiometers; (2) Invar Wire; (3) Failure Surface; (4) Fixity

Fig. A5 Inverted Pendulum Type Inclinometer.

to pull one of the wires connected to the potentiometer; pulling apart of the wire causes the potentiometer to turn and thereby change its resistance value, which can be measured by a simple electric device. The equipment can be made automatic by deploying servo a accelerometer in place of a potentiometer.

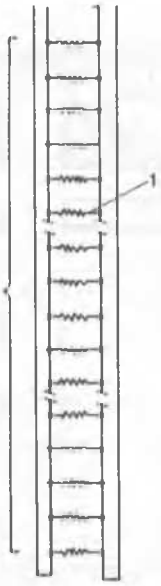
Blondeau, Failure Strips (Fig A6)

An nrray of the known value of resistances are connected in parallel on a brittle conduction tape and placed inside a plastic tube and subsequently into a bore hole. The displacement in the shear zone causes the conduction tape to shear. The location of the shear zone can be detected by simple electrical measurements. The accuracy of detection of the shear zone depends largely upon the spacing interval of the resistances.

Hickl's Strip Inclinometer (Hickl, 1977, Fig A7)

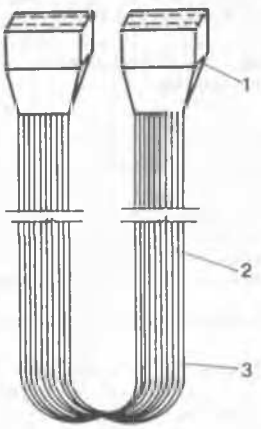
A flat U shaped cable having many parallel conductors, interconnected at regular intervals with thin conducting electrical wires, is located in a plastic tube which is then filled with cement grout and the whole set up is installed in a borehole.

The bridges of the conducting electrical cables snap on account of displacement, for example, near the shear zone. The location of shear zone can therefore be established by making electrical measurements. Exact time of snapping can be found by coupling an alarm system with it. False readings, due to the maintained continuity des-



(1) Resistances (n resistances R_0)

Fig. A6 Blondeau, Failure Strips.



(1) Connector for connecting the checking device

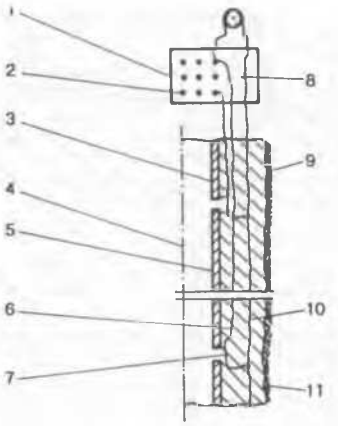
(2) Connecting bridge (3) Strip conductor

FIG. A7 HICKL'S STRIP INCLINOMETER (HICKL, 1977).

pite snapping of cable bridge, because of the local electrolysis of the water, may create interpretation problem.

Failure Detection Tube (Developed by Hydraulic Engineering Research Institute, Romania, Fig A8)

The failure detection tube developed by the HERI, Romania is similar in principle to that of Hickl's Strip inclinometer. The only difference is that instead of a flat cable, independent electrical cables are installed in the borehole. The cables snap at the shear zone and this can be established by simple electrical instrument. The detection can however only be made after the

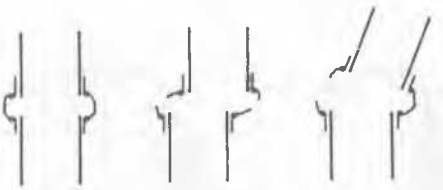


(1) Apparatus for circuit continuity checking

(2) Checking panel for „d-e-f" circuits

(3) Terminals for separate

(4) Borehole axis



(1) Apparatus for circuit continuity checking

(2) Checking panel for „d-e-f" circuits

(3) Terminals for separate „d-e" connections

(4) Borehole axis

(5) Sectioned pipe (50...100 cm sections)

(6) Separate current-carrying conductor for every wire

(7) Wire with maximum displacement max

(8) Terminal for the „f" connection

(9) Borehole wall

(10) Current carrying conductor for all „d" wires

(11) Plastic grout sheath

FIG. A8 FAILURE DETECTION TUBE (DEVELOPED BY HYDRAULIC ENGINEERING RESEARCH INSTITUTE ROMANIA).

displacement in the shear zone is greater than the snapping limit of the cable. An alarm can be provided to indicate the time of the breaking away of the cable. There should be some mechanism to avoid false continuity of the cable due to the local electrolysis of the water in and around the soil.

Electrolevel, Ground Deflection Measuring System (Fig A9)

The system deploys the use of electrolevels to measure ground tilt, which can be monitored or recorded by a bridge circuit. The instrument comprises a rectangular steel block with electrolevel system attached to it. A Steel tube is fixed in the ground. On the top of the steel tube is placed an accurately ground cadmium plated platen with four reference lines each at 45°. The electrolevel attached to the steel rectangular section is placed repeatedly on the cadmium plate to obtain error free result. One electrolevel can be used to monitor several locations.

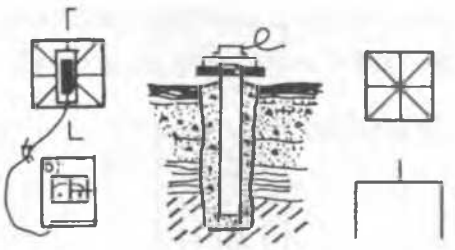


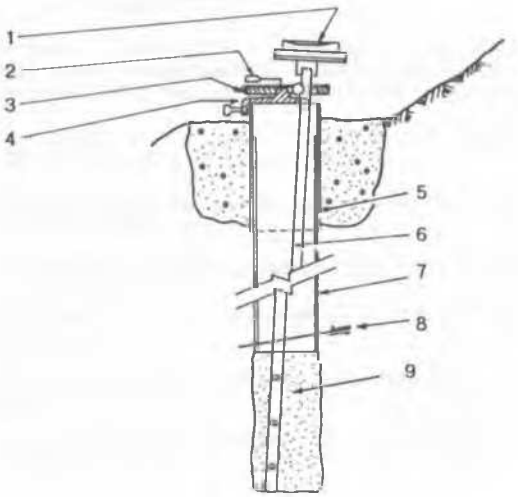
Fig. A9 Electrolevel Ground Deflection Measuring System.

Shear Detection Using Tiltmeter (Sherwood and Currey, 1974, Fig A10)

A cantilever rod is anchored below the level of a slip surface. It bends with the movement of a surface reference casing. This is analogous to point loading of a cantilever and, from bending theory the horizontal displacement (d) at the point load is related to angular rotation θ at that point. An electric sprit level is deployed to determine the subsurface movement.

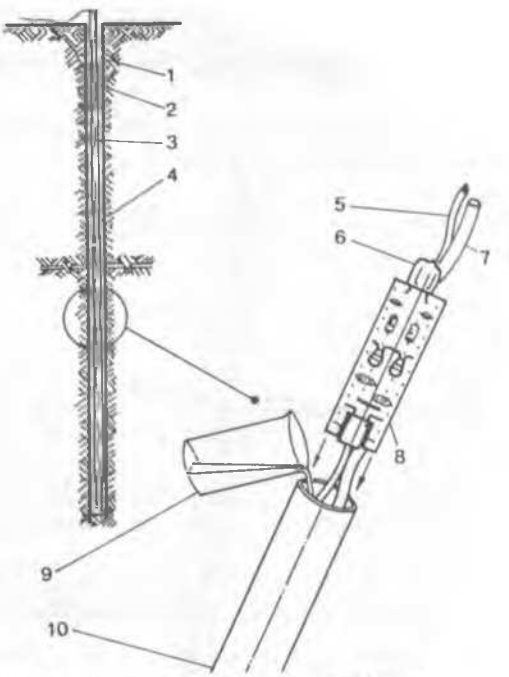
Fixed in Place Inclinometer (Enever et.al.,1977, Fig A11)

A string of electrolytic bubbles complete with electronic circuitory is encapsuled in short rigid tubes and the required number of such short tubes are converted into a string using multi-cables. The units are assembled in a flexible pipe to allow orientation of the units with the single active direction of measurement. It has range of $\pm 15^{o}$ and sensitivity of 2 minutes of an arc.



(1) Electric spirit level
(2) Dial gauge
(3) Adjuster
(4) Locating bridle
(5) Surface reference casing
(6) Cantilever rod
(7) Flexible casing
(8) Shear plane
(9) Cantilever rod anchorage

FIG. A10 SHEAR DETECTION USING TILTMETER (SHERWOOD AND CURREY, 1974).



(1) Flexible joining lengths (IC PVC)
(2) Grout
(3) Cables (power & signal separate)
(4) Rigid inclinometer unit
(5) Power cable
(7) Signal cable
(6) Bubble tube
(9) Epoxy resin
(8) Demodulator circuit
(10) Rigid casing

FIG. A11 FIXED IN PLACE INCLINOMETER (ENEVER ET. AL., 1977).

Fixed Inclinometer,Vibrating Wire Type (Multiple Deflectometer Manufactured by M/s. Interfels, Fig A12)

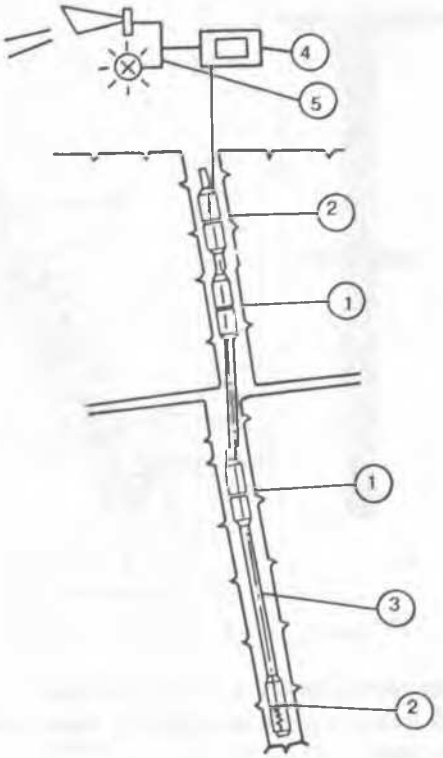
Comprises a multiple unit of vibrating wire type transducers which are interconnected by steel wires and spacing tubes. The wires stretch in response to displacements. Consequently a change in the tension and therefore of the natural frequency of the vibrating wire in the transducers. The change in frequency is measured by a frequency counter and is eventually converted in terms of slope movement using calibration curves of the instrument.

Servo Accelerometer Type Inclinometer (Schaevitex, 1976, Fig A13)

A paddle attached to the inclinometer tends to move as the inclinometer is tilted from its original position. The force exerted by the servo accelerometer in restoring the paddle to its original position can easily be measured by simple electrical circuitory.

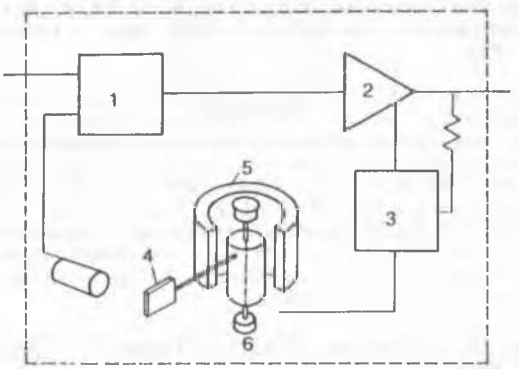
Fixed Inclinometer (Vibrating Wire Pendulum Type Telemac Model MP100, Fig A14)

The Inclination is measured by monitoring the movements of the free pendulum. A differential



(1) Electrical Transducer; (2) Head and End Piece with Wire Tensioning Device; (3) Wire between Deflectometer Elements; (4) Electrical Readout Unit with Limit Switch; (5) Optical or Accoustical Warning Device.

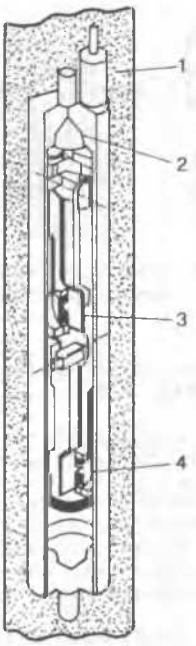
Fig. A12 A Multiple Deflectometer.



(1) Electronics Module; (2) Servo Amplifier; (3) Damping; (4) Paddle; (5) Torque Motor; (6) True Vertical.

Fig. A13 Servo Accelerometer Type Inclinometer.

inductance transducer is used to monitor the movements. The variance in the inductive transducers can then be measured electrically. This is a very sensitive instrument & reads accurately to 10^{-6} rad over a span of $\pm$ 1.1°.



(1) Grout (3) Pendulum
(2) Electronic module (4) Coil

FIG. A14 FIXED INCLINOMETER (VIBRATING WIRE PENDULUM TYPE TELEMAC MODEL MP100).