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Road Construction in Soil Creep Areas

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ABSTRACT: This paper presents a general view and framework of engineering construction over ground with characteristics of creeping deformation and addressing the driving forces and probable causations of such observed creeping deformation. The common problems or engineering implication arising from such creeping deformation to linear infrastructure development will be discussed from the aspects of design, construction and maintenance. Strategy and possible engineering solutions for each nature of the creeping deformation are summarised. Ten interesting case histories with creep movement will be briefly presented to illustrate the investigative findings.
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1.0 INTRODUCTION

The generic definition of creeping deformation means continuous deformation of materials without any changes of the stress within the subject materials. However, perception of creeping deformation has perceived as continuous straining of materials without concerning the stress changes. Sometimes the scale of the affected area or the time scale of occurring deformation have tendency leading to general conclusion of material creeping rather than understanding the actual mechanism.

This paper will attempt to explain the creeping phenomena of founding materials in relation to the common engineering problems. The resulting outcome from materials subject to creeping and the potential strategy and engineering solutions to dealt with creeping phenomena as perceived by engineers will be discussed. Some relevant case histories with "creeping characteristics" will also be included to illustrate the actual behaviours of the perceived creeping movement of ground.

2.0 NATURE OF CREEP AND ITS FACTORS

From observation in material laboratory or even simple experience a layman can perform on a rubber band with a hanging object, creeping can be easily observed with the continuous elongation of the rubber band with time without increasing the weight the hanging object. Simple observation of material creeping is the deformation of the observed materials. In strict term, creep deformation seldom involves failure of the material that creeps. The deformation of creeping materials is basically the summation of the occurred plastic straining within the materials. If deformation involving localised ruptures within the material is considered as the outcome of creep, then there will be many other factors, which can lead to the observed deformation. For the purpose of this paper, it will be categorised the creep soil movement into two group, namely (a) straining creep, and (b) rupture creep involving discontinuous sliding along or gap opening at ruptured surface.

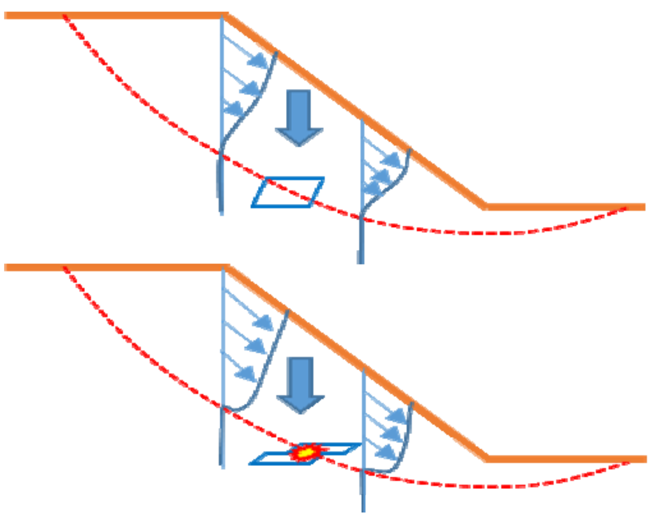

Figure 1: Straining Creep (Top) & Rupture Creep (Bottom)

Cruden and Varnes (1996) had classified the velocity of the landslide movement as in Table 1. Most laypersons likely perceive creep movement with velocity class 1 to 2.

Table 1: Terms describing the velocity of a Landslide (Cruden & Varnes, 1996)

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

All materials have deformability and strength to withstand a certain level of stress. When the material subjects to larger stress, the more immediate elastic or even plastic strain (if yield point is reached) can occur. Until the induced stress within the material exceeds its strength limit, localised rupturing will occur at the area where the strength is exceeded, and the unbalanced stress at the ruptured area will then be redistributed to other adjoining soil elements. The propagation of rupture may continue till the total failure of the material if the destabilising forces resulting from either load imposition, increase of pore water pressure or reduction of material strength after failure persists. Figure 2 shows diagrammatic stress displacement relationship of both low and high clay fraction soil samples with a shear surface at constant normal stress. It is not difficult to observe that the strain softening and dilation (resulting increasing water content in the sample) during shearing stage, the strength of soil sample with low clay fraction will stabilise after reducing from higher peak strength to lower critical state strength (fully soften strength). However, with continuous shearing, the strength for soil sample with high clay fraction will further reduce with continuous shearing where the clay particle re-orientates resulting much lower residual strength. It is also known that over consolidation of high clay fraction soil will result higher peak strength and more brittle behaviour in strength reduction from peak strength to critical state strength and then to residual strength.

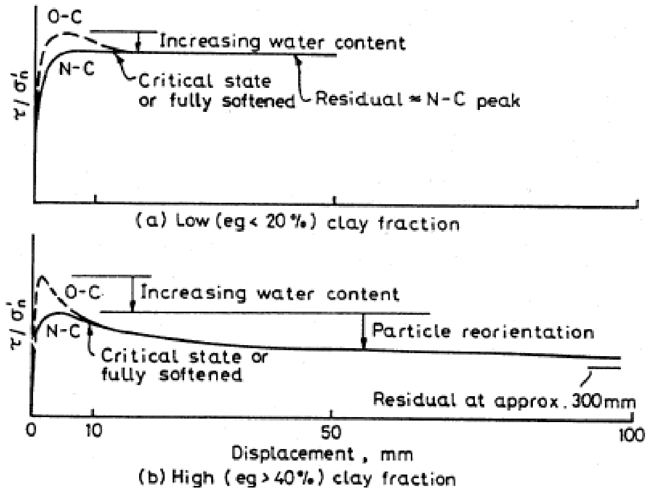

Figure 2: Stress displacement curves at constant normal stress σ_n' (Skempton, 1985)

For a material mass to move or deform, there will be a force or stress to induce the movement or deformation. The most common force on the earth is gravitational force. Other natural forces affecting the earth crust would be the tectonic stress as a result of the convective flow of molten mantle materials beneath the earth crust as shown in Figure 3. The earth crust will subject to compressive stress at colliding trench and tensile stress at the ridge (normally at ocean). The compressive thrust pushes up ocean deposits forming mountain and continental land. At the same time, rupturing of the earth crust resulting from huge tectonic stress creates faults (when its strength limit is exceeded), foldings and other geological structures. There are three common types of faults, namely normal fault, reverse/thrust fault and lateral strike-slip fault as shown in Figure 4. The existence of tectonic stresses within the earth crust will intermittently strain the crust to form folds and rupture earth crust to form faults with quake vibration.

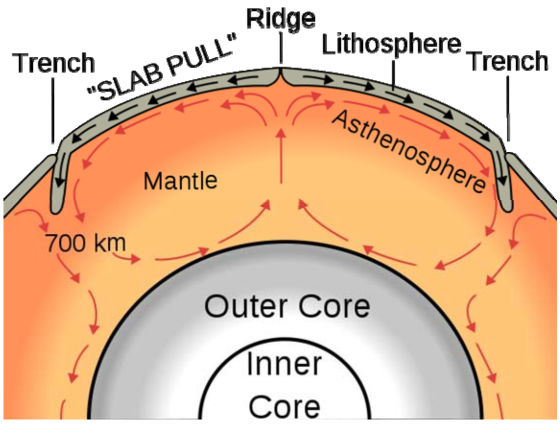

Figure 3: Diagrammatic earth crust with convection of molten mantle (Wikipedia)

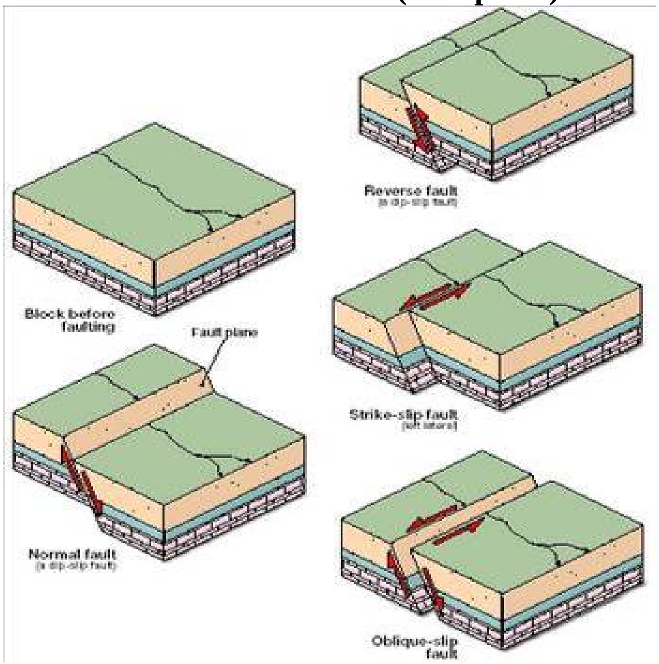

Figure 4: Typical types of Faults with relative ground movements (www.scienceclarified.com)

The acting of forces from either one or combined would cause deformation of the surficial formation on the earth crust. Vertical gravitational force will reduce the potential energy of an earth mass from higher ground to a lower ground. The work done from this change of potential energy is to optimally rupture the continuous earth mass into a contact surface to facilitate sliding of detaching earth mass at varying movement rates in Table 1. This phenomenon is commonly termed as landslide, or debris flow for a mass movement with higher mobility. Sometimes, a massive landslide events can be a collective outcome of retrogressive or progressive type of separated local slide failures in serial as shown in Figure 5. Retrogressive failure refers to a situation where lower part of a slope fails, thus removing the support from this part of the slope to the upper slope, and subsequently results in the successive failure extending into a region where previously the factor of safety was greater than one. Whereas progressive failure is a situation where the soil (or rock) is strain weakening, and this results in area of high stress in a slope reducing in strength as the soil yields (localised failure) with the stresses in the slope redistributing to adapt to the changed yield strength. To have progressive failure, it is necessary to have non-uniformity of shear stresses, and boundary conditions such that strains exceeding failure may develop. This may progress through to collapse of the slope. The redistributed stress from previous failure of adjacent earth mass may over stress the adjoining earth masses, which may be at subcritical stability condition leading to domino effect of collapsing the entire slope. In some cases, the instability can be triggered by excessive cutting leading to unbalanced stability of the marginally stable slope. Once the instability is triggered over certain inherently weak geological features, it would be costly to remedy.

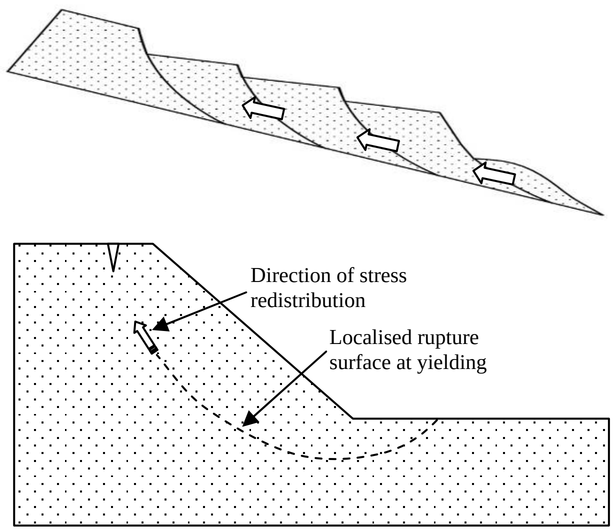

Figure 5: Retrogressive failure (top) and Progressive failure (bottom)

Leroueil et al (1996) conceptualised the slope movements of sliding type into four different stages as illustrated in Figure 6.

Pre failure stage – When the slope is strained throughout, but still essentially intact without full development of complete rupture, however, localised rupture may occur within the soil mass.

Onset of failure – this is characterised by increasing rate of movement during the formation of a continuous rupture surface immediately at failure.
Post failure stage – gradually reduced rate of movement of slope mass after failure until reaching marginally stable condition.
Reactivation stage – When the slope slides along one or more pre-existing shear surface in occasional or continuous manner. There may be differential movement of the instable slope masses. The rate of movement is some time dependent on the seasonal variation of the weather.

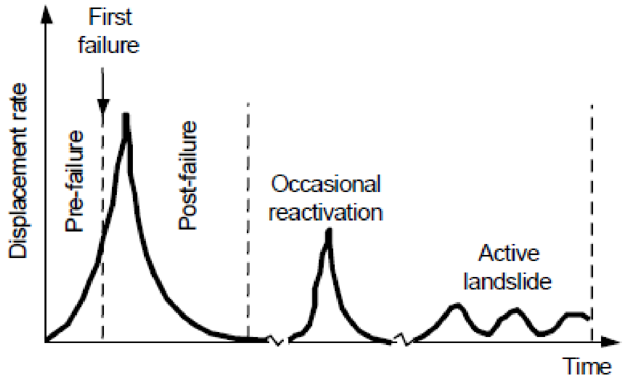

Figure 6: Different Stages of Slope Movements (Leroueil et al 1996)

Fell et al (2000) have given a very good summary of insight and ideas of pre-failure creep movement of a first time sliding contributed from a group of researchers. The extract of the summary are reproduced below.

(a) At the micro scale of soil skeleton, creep movements are attributed to the breaking of bonds and the re-orientation of soil particles; visco-frictional sliding relating to the inter-particle contacts; pore pressure increases and weakening of the clay skeleton by a fatigue phenomenon (in undrained creep) in unstructured clays. There is also evidence that the effects of strain rate and temperature combine in a unique viscosity law, which could be explained by the rate process theory related to the activation energy at the level of molecules, atoms or particles. This theory is synthesised by Mitchell (1993).

(b) At the macro scale, Mitchell (1993) explains creep in over-consolidated clays in undrained loading as "shear deformations cause dilation and development of negative pore pressures, which do not develop uniformly throughout the sample, but concentrate along planes where the greatest shearing stresses develop. With time during sustained loading, water migrates into zones of high negative pore pressures leading to softening and strength decrease relative to the strength in "normal" undrained strength tests. Figure 7 shows the effect.

As explained by Mitchell, the effective stress path is represented by AB, and AC represents the total stress path in a conventional CU (consolidated undrained) test. The negative pore water pressure at failure is CB. If a creep stress DE is applied to the same clay, a negative pore pressure EF is induced. This negative pore pressure dissipates during creep, and the clay in the shear zone swells. At the end of the creep period, the effective stress will be as represented by point E. Further shear starting from these conditions leads to strength G, which is less than the original value at B. It is evident also that if the negative pore water pressure is large enough, and the sustained load is applied long enough, then point E could reach the failure envelope.

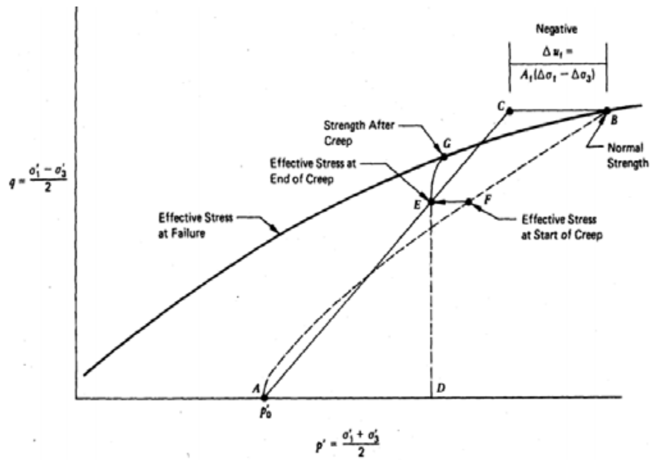

Figure 7: Stress paths for normal undrained shear and drained creep of heavily over-consolidated clay (Mitchell 1993).

(c) It has been evidenced that the strength envelope of clayey soils in their over-consolidated domain is time or strain rate dependent. Figure 8 also provides evidences the fact that the pre-consolidation pressure is strain rate dependent. Creep failure in this domain thus results from the combined effect of a lowering of the strength envelope and change in pore pressure. On the other hand, there is no significant strain rate effect on the normally consolidated friction angle, so that a reduction of undrained shear strength with time or decreasing strain rate in this domain is associated only to pore pressure increase. These phenomena results in an undrained shear strength that typically decreases by 10% when strain rate decreases by one order of magnitude (Figure 9).

(d) These viscous phenomena are now well recognised for laboratory conditions. However, the links with practical applications are almost not existent. In particular, the rate of testing is not discussed in these terms and would certainly be an interesting aspect for research.

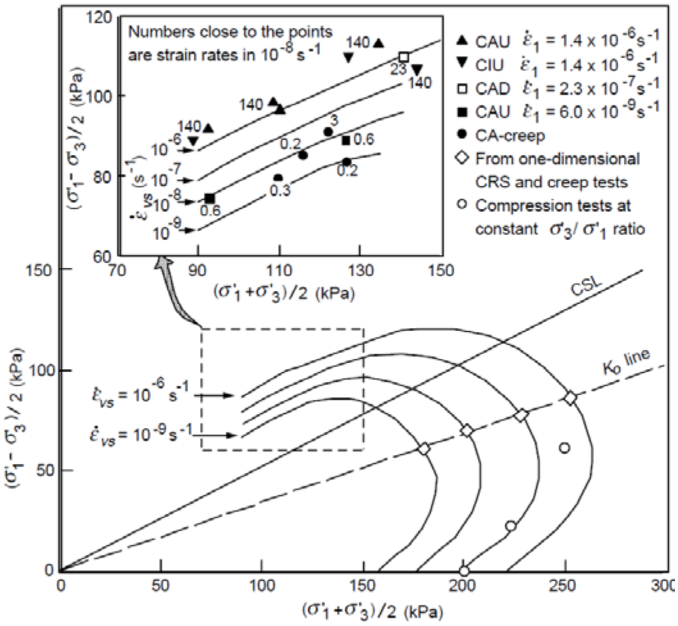

Figure 8: Influence of strain rate on the limit state of Mascouche clay (Leroueil and Marques, 1996)

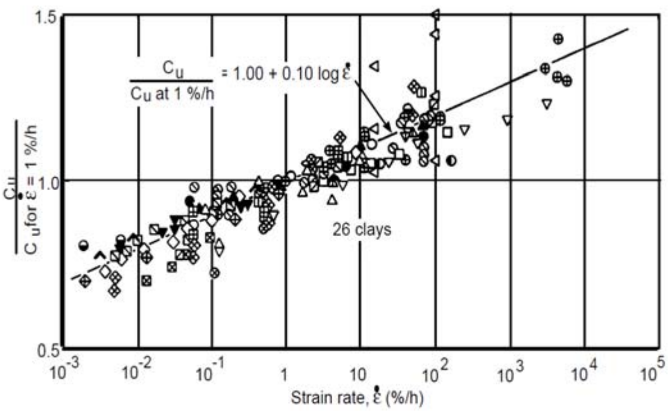

Figure 9: Influence of strain rate on the undrained strength measured in tri-axial compression (Kulhawy and Mayne, 1990)

Some general characteristics of creep behaviour are:

(a) Creep deformation exists within all soil elements regardless of any levels of deviatoric stress. The stress conditions of soil elements within a "perfectly" stable slope are always in the over-consolidated domain, i.e. within the yield surface of the t-s stress plot. Figure 10 shows schematically the stress conditions in a saturated slope. The state of stress for a stable soil element can be represented by a point such as D in Figures 10a and 10b. If the slope is subjected to either loading at the crest or erosion or excavation at the toe (stress relief condition), the stress conditions move respectively to L or E on Figure 10a. As the groundwater regime in the slope varies seasonally, the stress conditions move in Lw direction corresponding to low water conditions or in Hw direction when high water conditions prevail (Figure 10(b)). Creep rates also vary with the seasons, being much higher when water conditions are high.

Local failure as initiated when the stress state, either E on Figure 10(a), or Hw on Figure 10(b), reaches the peak strength envelope corresponding to the age or the strain rate of the slope. After surpassing the peak strength, stress conditions progressively move towards the critical state (CS) line, and the unbalanced shear stress of the stressed soil element being limited to the critical state strength (lower than the peak strength) is transferred to the neighbouring soil elements, in which the local failure would be progressively extended to. This phenomenon of progressive failure can extend into a continuous development of shear surface through the entire soil mass, and then trigger a landslide when the ruptured shear surface extends throughout the entire soil mass.

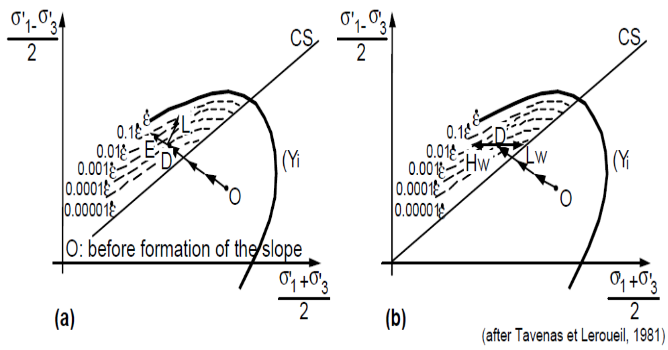

Figure 10: Schematic creep effects and effective stress conditions in natural slopes
(a) For loading at the crest (DL) and erosion at the toe (DE), (b) for pore pressure fluctuations,

DHW and DLW (Tavenas and Leroueil, 1981 Leroueil et al 1996)

(b) The process of creep can be defined as occurring in three stages as shown in Figure 11; primary (at a decelerating strain rate), secondary (at a constant strain rate), and tertiary (at an accelerating strain rate), generally to creep rupture or failure. However, the secondary creep can be a short transition zone bridging over the primary creep with decelerating strain rate and the tertiary creep with accelerating strain rate.

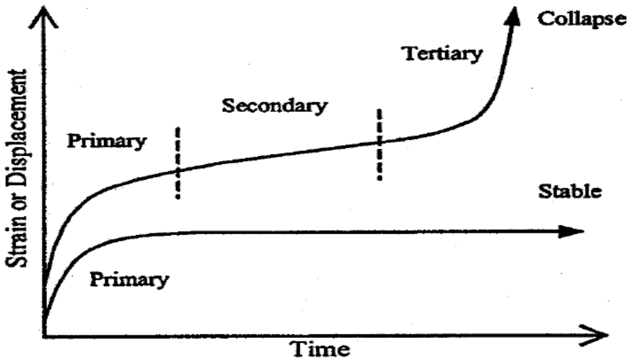

Figure 11: Diagrammatic representation of creep for moving slopes

(c) Soils subject to creep will fail at deviatoric stresses even less than the peak strength. Leroueil (1998) and Hunter and Khalili (2000) highlighted the following salient points.
- Creep to failure can occur at less than peak strength with the limiting strength possibly being as low as the fully softened (critical state) strength (Figure 12(a)). Hence, it would be sensible to practically base the design strength at critical state strength to reduce the risk of having creep failure.
- The level of shear strain at which the onset of failure due to creep occurs is equivalent to the shear strain at peak stress (ε_{peak}) in the equivalent conventional strength test. The onset of failure is defined as the point of minimum strain rate, the change from primary to tertiary creep behaviour.
- The time to onset of failure can be predicted based on the accumulated shear strain (Figure 12(b)).

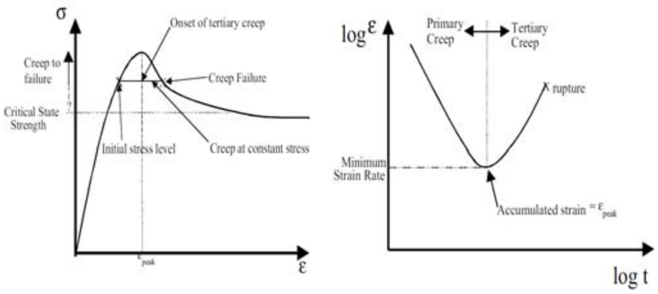

Figure 12: (a) Idealised creep to failure at Stresses Less than peak strength (Left), (b) Onset of tertiary creep when accumulated strain ≈ ε_{peak} (Hunter and Khalili, 2000)

As it has been demonstrated earlier, the role of pore water response within the soil skeleton is vital for the stability of the earth, in which the variation of pore water will affect the stress state or stress paths movement within the failure envelope of strength, and also the rate of creep straining. Hence groundwater regime and its variation can certainly be an important triggering factor for instability.

For ground movement induced by tectonic stresses, the extent of the movement zone are usually extensive both in vertical and horizontal directions. Similar to the gravity induced ground movement mentioned earlier, the deformations involves continuous straining of the intact earth material without rupturing the material and also discontinuity in deformation with rupturing when material strength limit is exceeded. As the tectonic stresses are usually huge and often exceed the material strength, hence ruptured earth materials are not uncommon at tectonic active zone. The continuous build-up of tectonic stresses within earth crust by active tectonic processes will find most easy ruptured interface for releasing the straining energy intermittently, thus active intermittent sliding of the ground can be evidenced from the observed sudden distresses on structures constructed on top of the movement zone. Sometimes, gaps or localised stress relief at the tectonic zone may create a conducive condition for subsequent creep movements under gravitational force. The hill side after quake attack can be loosen forming marginally stable detached earth blocks, in which sliding can be triggered after partial saturation with water. There were more reported landslides and debris flows at Taiwan after the 921 quake incident in year 1999. Same observation also applies to the recent earthquake on China.

There is another type of deformation commonly found in civil engineering application, which is secondary creep of materials under compressive loading. For example, when the fill platform placed over organic materials, likes peaty soils or peats, consolidation compression with pore water dissipation and secondary creep compression can occur over a long period of time. This will result in observable settlement on the platform, where the compression is mostly contributed from the underlying compressible deposits. When there is a structure with piled foundation, contrast of settlement profile across of the settling platform and the structure with relative less settlement can be a problem if smooth continuity is expected in the design. The mechanism of creep movement of this nature will not be covered in the scope of this paper.

3.0 COMMON ENGINEERING PROBLEMS OF CONSTRUCTION WITH CREEP DEFORMATION

The action of the gravitational force to a sloping earth mass is usually static in nature until the strength of the earth mass is exceeded, where the slide movement along the ruptured surface can vary from very slow creeping movement to high mobility slide movement. However, there is no established method to assess the rate of such instability movement as the factors contributing to such assessment is overly complex, hence has not been practically available. Presently the norm of practice in engineering assessment is to assess the adequacy of factor of safety based on the geometrical conditions, hydro-geological conditions and the investigated engineering parameters of the slope materials. This can be done by conventional limit equilibrium analysis (LEA) or strength reduction method in finite element analysis (FEA) on stability. However, this method is to address the stability problem on ruptured creep as mentioned earlier.

Tongkul (2006) has emphasised the importance of geological influence on the slope failures in the mountain area. In Sabah, due to the NW Borneo fold-trust belt formed at west coast, some important inter-district roads are experiencing reoccurrence slope instability with costly maintenance as these roads are passing through Crocker Range with unfavourable orientations of discontinuities and slope face cuttings. He also listed several main reasons of failures that such slopes are normally located at:

a. a localised fault zone;
b. with unfavourable orientation of discontinuity planes;
c. located at a regional fold hinge or
d. at an old landslide deposit.

For areas affected by large scale tectonic disturbance, it would be prudent to avoid any construction over the affected area. Currently there is no simple approach to deal with massive creep movement triggered by tectonic activities as the depth and extent of affected area are for too extensive for the necessary strengthening with practical cost of investment.

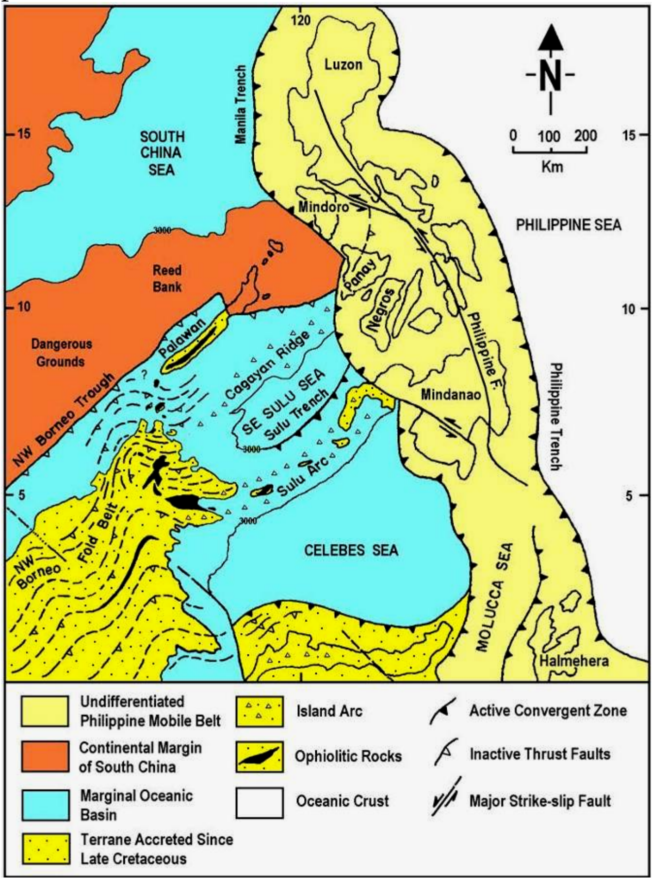

Figure 13: Tectonic Setting of Sabah showing the NW Borneo Fold Belt (Tongkul, 2006)

The ground deformation or even collapse will have the adverse impacts to any linear infrastructure development or localised building development. Common problems are summarised below.

- Entire or partial loss of platform
- Cover up with collapsed earth mass over the affected area or platform
- Distortion or tensioning to any rigid structures over deformed ground until suffering damages rendering unserviceable conditions
- Distressing or collapse of structures

- Leaking of fluid or pressurised gas carrying utilities
- Tensioning or buckling of pipelines, cables , ducts and tunnels
- Cracking of road pavement surface
- Shifting of structures beyond property boundary
- High frequency of maintenance to ensure uninterrupted service
- Uncontrolled runoff erosion failure at cracks between sliding blocks

4.0 CASE STUDIES

A total of nine case histories within Malaysia are selected with relevance to construction associated with creep movements to illustrate the observed behaviours and associated mechanisms, investigation approaches taken, findings of outcome and solutions to fix the problems. These nine case histories are summarised in Table 2. For details of each case history, readers are encouraged to download the papers from the link (http://www.gnpgroup.com.my/publication.asp).

From these case histories, it would be able to attain the following common grounds relevant to creep movements.

a. The back-analysed mobilised strength at the time of failure is usually lower than the interpreted peak strength. This is a feature of creep failure as highlighted by Leroueil (1998) and Hunter and Khalili (2000). Thus, it is advisable to adopt critical state strength for slope stability design to minimise the risk of creep failure except for remediating failed slope without removing collapsed debris, in which residual strength shall be appropriately considered.
b. Inclinometer is a very useful instrumentation tool to reveal the ground deformation with time to provide creep behaviour observation.
c. Loose fills have high tendency for creep movement when subjecting to load imposition or stress relief by toe erosion or excavation. Hence, proper compaction of placed fill will prevent failure within the fill.
d. In most earthworks, filling of valley to create usable platform is not uncommon in rolling terrains. Soft compressible materials deposited at the valley and the vegetation growth are a potential source for creep movement. If not properly removed, it will cause serious strengthening works required for fill materials placed on top of the unremoved soft compressible materials.
e. Natural valley is natural channel for accumulation of groundwater and also infiltrated water from the fill placed over it. Proper subsoil drainage is the crucial design element to control fluctuation of moisture within the fill.
f. Erosion failure at the downstream of drain discharge is also common if the collected surface runoff and subsoil drain discharge are not properly channelled to the natural stream. Necessary energy dissipation design detailing shall be considered if needed. Leaving the drainage discharge at the interface of the right of way (ROW) will not have the problem solved. When erosion occurred and detected, the remedial cost can be tremendous as most of the erosion failure occurs at fill embankment over the natural valley, where usually the drainage discharge will be located.
g. Sometime geological features can be a contributing factors for creep failure. It is always useful to conduct geological appraisal to the site to detect the potential geohazards. However, it would be a tough challenge to explicitly identify relict geological structures in completed weathered soils. But if the adverse geological structures can be detected at the nearby outcrops, it is reasonable expect that the same configuration of relict structures may exist in the weathered residual soils, which is derived from the same parent formation.

Table 2: Summary of Case Histories with Relevance of Creep Movements of Soils

Case History	Authors	Problem Statement	Findings and Solutions
1	Liew, S. S. & Gue, S. S. (2001)	This case study presents an investigation and instrumentation of a massive creep movement of the post-glacial deposits (Pinosuk Gravels) long a 1.2km road (Jalan Cinta) with gradient of 15.5% at Kundasang area, Ranau District of Sabah. There was very frequent maintenance needed for the pavement, drains and overhead cables as a result of the differential creep movement at the area.	Slip surface at 6m (higher elevation) to 15m (lower elevation) and direction of movement generally toward south-westward were detected from inclinometers. High groundwater table was also determined in the standpipes. The rate of slide creep movements generally within 15mm/week with few rare occasions of maximum 34mm/week. The back-analyses mobilised strength (c_m'=0kPa & ϕ_m'=15~16°) is much lower than the interpreted peak strength (c_p'=10kPa & ϕ_p'=21°) from Consolidated Isotropically Undrained (C.I.U.) Tri-axial tests with pore pressure measurement.
2	Liew, S. S., Gue, S. S. & Liong, C. H. (2003a & b)	This case study involves a well-documented failure investigation of a two-berm cut slope formed in colluvium of Jurong Formation at Johore area. Comprehensive instrumentation scheme was implemented in the post failure slope condition to investigate the continuous creep movement even after failure. High groundwater was also found in the failed debris as observed in the standpipes.	Multiple slip surfaces with continuous creep movements were detected in the inclinometers installed after the failure. The rate of slide movements sometimes appears responding to the major rainfall event with obvious intensity, but occasionally also to a minor rainfall events. The rate of movement started with initial 28mm/day reducing to 7 to 17mm/day at extreme cases during remedy, and stabilised after remedy. The back-analysed mobilised strength (c_m'=0kPa & ϕ_m'=14.4°) for the failed slope is lower than the peak strength (c_p'=3.5kPa & ϕ_p'=32°) and critical state strength (c_{cr}'=0kPa & ϕ_{cr}'=29°) from C.I.U tests implying approaching lower residual strength.
3	Liew, S. S. (2004a)	The edited chapter consists of six case studies. Sites A & B are in fact Case History 1 & 2 in this summary. Site C involves a progressive failure of a 45m high multi-berm cut slope with top seven cut berms of 1V:1H, and lower five berms of 4V:1H, which were strengthened by soil nails and shotcrete at Gua Musang, Kelatan. The site formation is Gua Musang formation enriched with day-lighting relict geological structures.	Site C: Finite element analysis (FEA) with strength reduction technique was used to investigate the failure mechanism with simulation of stress relief from the cutting, in which progressive failure with initiation of localised yielding of soil elements at different locations was clearly revealed during the successive slope berms cutting and installation of soil nails. The day-lighting relict geological structures also contributed to the failure as it is believed that the development of localised yielding may couple with the relict geological structures to develop a kinematic permissible failure. It is also confirmed that the ruptured surface is beyond the installed soil nails rendering no contribution of the nails to resist the failure.

4	Liew, S. S. (2004a)	Site D involves a slope failure of a six berms cut slope of total height of 35m at Kuala Lumpur Granitic formation. In front of the failed slope, there is an apartment with its perimeter drain collapsed by the failed slope toe. The slope materials is fairly stiff weathered residual soils derived from the granite. The failed slope still remained intact as a rigid mass detaching from the original position.	Site D: The inclinometers show a ruptured surface assembling a circular slip surface. High groundwater table was recorded in the standpipe installed near the slope toe. The interpreted strength from C.I.U. samples indicate the peak strength: $c_{p,lb}$'= 0kPa, $\phi_{p,lb}$'=27° (lower bound); $c_{p,mc}$'= 2kPa, $\phi_{p,mc}$'= 31° (moderately conservative); $c_{p,ub}$'= 5kPa, $\phi_{p,ub}$'= 39° (Upper bound). From the back-analysis with the detected circular slip surface and the recorded groundwater table, it was clear that computed factor of safety against failure is near 1.0. Finally soil nailing strengthening technique with pads has successfully remediate the failure.
5	Liew, S. S. (2004a)	Site E is a failure case of a three-berm 1V:1H fill embankment over a natural valley to support a gas utility pipeline at Salak Tinggi of Selangor. The site geology is Kenny Hill formation, in which the fill materials were believed to be the weathered derivatives of the same formation. This fill embankment has been reconstructed twice before the failure. The embankment platform has experienced creep movement affecting the safety of the gas pipeline. The failure happened during heavy rainfall event. From site observation immediately after failure, it was concluded that there were significant amount of runoff flowing through the fill embankment, which has possibly saturated and soften the fill embankment.	The interpreted peak shear strength from the C.I.U. tests is c_p'= 2kPa, ϕ_p'= 32°. Back analysis was carried out on the slope profile of the three berms slope to estimate the mobilised shear strength during failure. It was found that when groundwater level rises near to the ground surface, the Factor of Safety of the filled slope geometry against failure with the interpreted peak strength parameters was only approximately 1.0. Finally the failed embankment is remediated with a fill embankment comprising of rock toe, seven berms of slopes with 5 m berm height and a gradient of 1V:2H. Proper subsoil drainage (French drain), internal drainage (drainage blanket) of fill embankment and surface drainage were provided to prevent build-up of internal pore pressure and surface erosional failure respectively. The discharge of the collected runoff and subsoil drains was channelled to a soak away rock mattress at the flat terrain at the upstream of an existing stream.
6	Liew S.S., Liong C. H. & Low C. L. (2004b)	The third case study in the paper was a 27m high five to six berms 1V:1H cut slope failure of road construction in a granitic formation at Kupang, Kedah. The first failure of stretch of 50m initiated two months after slope formation, then followed by a larger failure of 250m in length in a heavy rainfall event in the third month. Comprehensive instrumentation scheme was implemented yielding useful information to conclude the causation and remedial design.	Slip surface was also detected by the inclinometer installed in the center of the slope. Observation wells indicated that the piezometric level at the slope toe was 0.37m to 2.2m above ground level. The interpreted peak and critical state strength parameters are c_p' = 2kPa, ϕ_p'=30° and c_{cr}'=1.9kPa, ϕ_{cr}'=28° respectively. Back-analysed mobilised shear strength are c'_m=0kPa, ϕ_m'=30° which agrees well with the C.I.U results. If the C.I.U results were adopted in a limit equilibrium analysis, the Factor of Safety would be less than unity, even without the

			effect of piezometric level. Therefore, the temporary stability of the slope after formation could be attributed to the existence of soil suction. The high rainfall record evidenced to contribute increase infiltration and the rise of piezometric level in cut slope, thus reducing soil suction and effective shear strength, and triggering the failure. Geological structure discontinuities seemed to be involved in the first localized failure event.
7	Liew S.S. & Khoo C. M. (2006 & 2007b)	This case history started with observation of pavement distresses (settlement and cracking) of a road on filled ground with adjacent mixed development involving earthwork cutting for building platform formation to accommodate a five storey car park below the road level. The area of distresses was found to be underlain by previous natural valley.	Investigation revealed that soft deposits at previous valley was not removed prior to fill placement. Huge amount of groundwater was accumulated at the valley area as revealed from the standpipes. The monitoring of the standpipes also reveals hydraulic connectivity of the groundwater within the valley to the surface infiltration during the rainfall event. Inclinometers installed also shown continuous lateral creep movement at the 2m thick soft deposits at the valley area when the platform cutting continued. In order to allow the sub-vertical excavation face for the car park, strengthening strategy using 6 and 12m soil nails with shotcrete facing and long horizontal subsoil drains to lower groundwater within the unstable saturated granitic loose fill was adopted. At the deep valley area, 12m long sheet pile wall and two rows of soil nail anchorage was used to enhance the passive resistance of the retained ground supporting the distressed access road. However, the loose fill resulted large creep movement for the soil nails during the pull-out tests, which the nail capacity has to be downgraded for safe design. The distresses was not arrested during the soil nail strengthening, because the drilling of soil nails in open holes will result in more ground loss, thus more ground movement. The ground movements become stabilised immediately after completion of the stabilisation works. FEA provided a very good insight of development of shear band within the marginally stable soil mass and allow proper provision of the anchorage length of soil nails to control the overall ground deformation.

8	Liew S. S. & Tan S. K. (2007a)	This case history consists of design and construction control of a 10m high reinforced soil wall for an access road over a thick soft compressible deposit at the natural valley on Kuala Lumpur Granitic formation. It was expected that some form of ground treatment required to support such heavy RS wall structure without using pile foundation. Stone column improvement was adopted. There was still concern on the potential lateral displacement and vertical settlement of the improved founding subsoil.	The instrumentation scheme deployed to monitor the concerned ground movement at the improved founding subsoil reveals rather acceptable settlement (maximum of 115mm) and lateral displacement (33mm) as against the design limits of 250mm to 280mm and 54mm respectively. The initial rate of lateral displacement of the improved founding subsoil fluctuated in the range of 0.2 to 1.25mm/day during wall construction, but stabilised with negligible rate of displacement in post construction stage.
9	Liew, S.S. & Khoo C.M. (2008)	Case history B in this paper presented an investigation of distressing condition to the platform of an automobile 3S (Sales, Service & Spare Parts) centre. The platform for the 3S centre is a filled ground over previous natural valley, in which the adjacent lot planned to develop an office tower with two levels car park below the platform of the 3S centre. A row of 12m long temporary sheet piles with a permanent Contiguous Bored Pile (CBP) wall was designed by the design-and-build contractor to facilitate the car park excavation. Distresses of settlement and cracking on the car park area around the 3S centre were noticed during earth excavation of the adjacent lot. Even the temporary sheet pile and CBP wall had deformed laterally about 615mm maximum (on top of the valley).	From the subsurface investigation, there was a layer of 6 to 9m thick very soft to soft sandy/silty clay at 7m below the platform the 3S centre, which is the probable causation of the platform distress at the 3S centre. The immediate temporary remedial solution was to backfill the excavation with stabilising berm of 1V:1H in front of the CBP wall. Two levels of internal struts propped against the partially completed car park basement structure was adopted to facilitate the intended excavation. The temporary sheet pile wall was lengthen to 18m to provide the necessary passive resistance.

10	Liew, S.S., Lee, S.T. & Koo, K.S. (2010a)	Case study B presented at creeping failure of an approach piled embankment to a river bridge for a highway. The creeping of the piled embankment successively imposing loading onto the piled abutment structure causing both tilting and distortion of the abutments on both sides. The tail ends of the piled embankment was seated on the improved ground with prefabricated vertical drains (PVD) and surcharging technique. The abutments were supported by front rows of raked piles and back row of vertical piles of both 400mm spun pile. Distress was first observed at the embankment on PVD treated ground, following by tension cracks on and sudden collapse of embankment on EVD treated ground. Thereafter spalling of concrete and gap opening at the bridge deck at Abutment B was observed. The rubber bearings at both abutments were founded distorted. Monitoring of bridge deck confirmed global movement from Abutment B towards Abutment A with slight clockwise rotation on plan view. There was a gap of 400 to 1000mm beneath the piled embankment slab implying consolidation settlement. Flexural cracking on piled embankment slab and the supporting reinforced concrete piles were noticed.	The subsurface investigation revealed that the site is underlain with very soft compressible clayey deposits. The back analysis on the failed embankment side slopes at the EVD and PVD treated areas only improved the original undrained shear strength by 2kPa from its original undrained strength of 10kPa. On the longitudinal direction, the stability analysis of the low embankment without pile support will impose lateral load to the piled embankment towards the abutment direction. With the free standing condition of the piles at piled embankment area, the lateral structural resistance from the piles was significantly reduced and was not able to resist the imposed lateral loading from embankment on EVD and PVD treated grounds. Hence the unbalanced lateral loading was thus transferred to the abutment, but being restrained by the bridge deck via the bearing contact. The flexural deflection of the pile beneath the embankment with the weight of the embankment will further jeopardise the stability condition as a result of the P-d effect. After concluding the causation of the bridge movement, the immediate action was to remove the earth embankment to relieve the lateral road on the abutments. The foundation piles and the embankment slabs for the embankment were reconstructed at lower platform to remove the previous temporary fill for piling platform, which caused the consolidation settlement leading to free standing condition of piles. The approach earth embankment at the abutment was detached with self-contained reinforced geotextile earth embankment with a transition slab connecting between the abutment and the approach embankment.

5.0 POSSIBLE OPTIONS OF ENGINEERING SOLUTIONS FOR GROUND WITH CREEP MOVEMENTS

Many solutions or conceptual ideas have been proposed to overcome the problems arising from creep movement. Options like tunnel and long span viaduct often come into the layman's mind to have the structures either above or buried deep into the ground with "surface creep movement". The subsequent sections provide a general evaluation to the said options and explored other special techniques for dealing with the creeping soils.

5.1 Tunneling

Tunneling through hill or mountain for road construction to achieve the desired vertical and horizontal geometry has been widely adopted all over the world. Tunnels are normally designed to be located in a "stable" formation under converging force from the overburden.

In formation with creeping problem, there will be significant unbalance force acting towards the tunnel which has high risk of collapse or dislocation. Such unbalance load can be massive and difficult to be estimated in the design due to limited data and studies. Firstly, it will not be surprised that the tunnel design would be structurally heavy as compared to normal tunnel design in stable ground. Secondly, constructing a tunnel into a creeping soil mass will first experience many construction difficulties as creeping soils mean unstable and problematic. Thirdly, the maintenance frequency is also expected to be high. In summary, tunnel option in creeping soils can cost tremendous capital investment and future maintenance, and no assurance of safety of the users. The rescue works for trapped vehicles or persons in tunnel are much more difficult.

An example of tunnelling in creeping soil: The city bypass tunnel of Waidhofen an der Ybbs (Austria) which has a section constructed over creeping soil (Adam, Markiewicz & Brunner, 2000).

Investigation of the area has been carried by installing inclinometers to determine the creep behaviour in the central section of the tunnel with an annual creeping rate of 14mm/yr, which is not too significant. Besides that, it is also identified that the residual shear angles determined on samples from this section, swelling clay minerals and relatively high natural water content are the factors to the instability of the slope.

First section of the tunnel in the creeping soil was constructed with cut and cover method. The adopted stabilisation works include improvement of drainage by installing slope gravel piles (similar to stone columns) which also to enhance the long-term slope stability. For the temporary work to support the excavation, soil nail and temporary pre-stressed anchors were adopted. For the second section where the NATM was adopted, they had to thicken their outer lining and the reinforced inner lining to reduce the deformation on the tunnel. Mitigation measure such as extensive slope doweling would be implemented if the ground movement showed adverse condition.

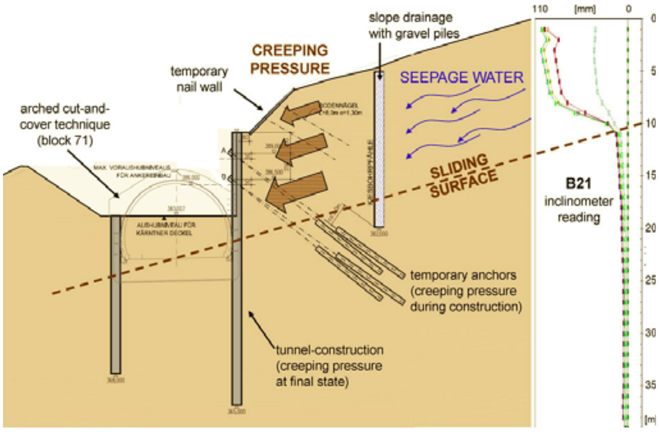

Figure 14: (a) Tunnel Section of the Cut and Cover Method (Adam, Markiewicz & Brunner, 2000)

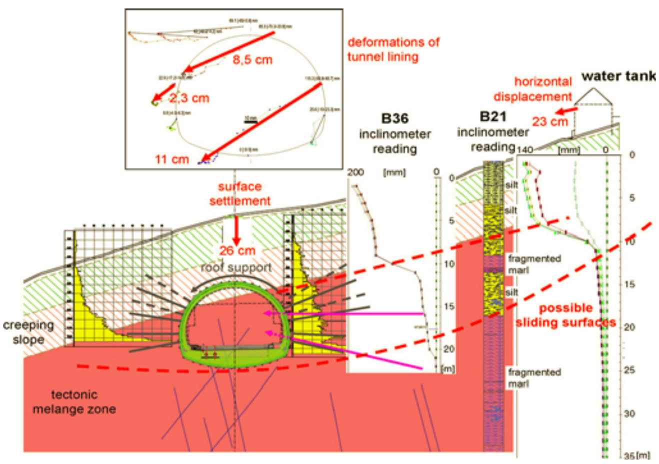

Figure 14: (b) Tunnel Section with NATM (Adam, Markiewicz & Brunner, 2000)

In all, constructing and maintaining a tunnel in a creeping formation involve massive cost. Therefore it is important to assess its financial feasibility. However, when the depth of the creeping soil is able to be identified and the tunnel alignment can fall outside of the creeping zone, tunnelling can be considered provided other options have been exhausted in feasibility stage.

5.2 Viaduct

Besides tunnelling, viaduct has also been proposed to minimise the disturbance to the natural slope. Similarly, it is important to identify the creeping condition before the option is adopted. For area with high creeping rate and great depth, the piers which are supporting the linear infrastructure will have to resist significant lateral load from the soil mass. Ideas of constructing a diverter and protection have been adopted in some countries. However, they are more like a preventive measure in the case when landslide occur. When the slope is actively creeping excessively, the allowance provided in the system will be utilised in no time and costly maintenance will have to come in again depending on the depth of creep.

An example of viaduct on creeping soil: The Beckenried Viaduct, Switzerland which has 3,150m long of viaduct traverses through an unstable and creeping slope (Vollenweider, 1984).

Altogether 44 of the total 58 piers of the bridge founded on sound rock had to be protected by shafts against the creeping soil and loosened rock layers. Each of these rectangular concrete piers are constructed together with an elliptical shafts which provide clearance of 1.5m in the dip direction of the slope and up to 1.0m in lateral direction. These flexible shafts consist of four main parts, namely a rigid shaft collar, articulated ring elements, a rigid trapezoidal cylinder and basal displacement rings resting on the pier footing which is to allow for the creeping and sliding. Besides that the shaft also act as a drainage pit to drawdown the water table in the slope as water from the slope is allowed to seep through the ring joints and being discharged subsequently.

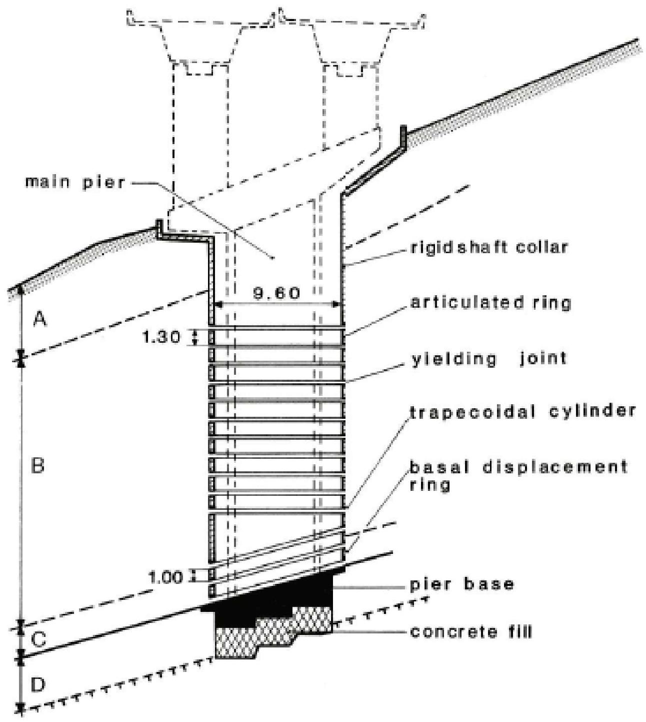

Figure 15: Typical Section of the Main Pier and the Shaft (Vollenweider, 1984)

Constructing road on viaduct is possible over the creeping soil area. However, when the rate of creeping is high and the affected stratum is deep, it can be too costly to be considered. On the other hand, when the creeping area or may be a fault line is identified, perhaps a long span cable bridge can be constructed to bridge over the problematic area or fault line. Such bridge will require flexible supports which frequent maintenance by adjustment and re-alignment work is needed. One condition to this cable bridge option is that the main piers must be on stable ground.

5.3 Dewatering Scheme

If the creep movement is associated with groundwater regime, dewatering scheme to lower the high perched water regime will help to enhance the soil strength and also reduce the driving force from water seepage to achieve sufficient factor of safety of the slope against creeping. For example, massive dewatering scheme has been deployed at Li-shan area in Taiwan in year 2003 to mitigate the creeping landslide (Wu & Su, 2007). Numbers of vertical caisson shafts (drainage wells) at pattern spacing were sunk into the creeping land as a collection sump pit for groundwater drawdown. Perforated horizontal subsoil drains were radial out from the caisson shaft at level where the design drawdown groundwater is expected below the sliding surface at 20m below ground surface. Drainage galleries was constructed to connect all the drainage wells at low invert level. Automatic level control submersible pumps are installed at the collection caisson shafts. When the groundwater is drawn down, the rate creep movement reduced drastically.

Another example is deep drainage method has been adopted in Campo Vallemaggia, Switzerland (Eberhardt, Bonzanlgo & Leow, 2007). A drainage adit has been constructed which has been effectively worked for deep-seated landslide that has been slowly moving for more than 200 years and involving estimated 800 million cubic metres of fractured and weathered crystalline rock with an expected sliding surface at up to 300m deep.

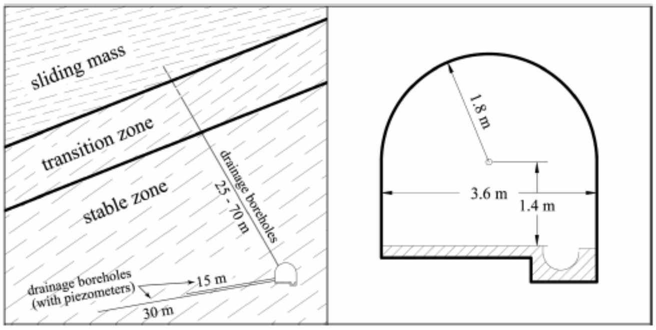

Figure 16: Schematic illustration of the drainage adit design with perforated cased drainage boreholes (left) and adit profile (right) (Eberhardt, Bonzanlgo & Leow, 2007)

However, careful studies on the potential environmental impacts arising from groundwater drawdown shall be performed. The potential impacts can be ground settlement, reduction of moisture content for agriculture lands, drying up of upstream creeks or rivers.

5.4 Other Flexible Structures

Conversely, rather than going against the nature force, there are options that accommodating to the creeping situation can be considered. Timber structure, which has high tolerable of distress due to its high flexibility structure, is probably suitable due to low cost of dismantling and reassembling of the structure. Comparatively to other massive structures, reconstruction or maintenance can be carried out more easily before the creeping effect is approaching to the serviceability limit.

Similar concept, light structure in "container" form can also be adopted which it can be adjusted and relocated as the ground creep provided the creep movement is not hazardous.

5.5 Mechanical Strengthening Scheme

If the extent of creeping can be identified, sometimes mechanical strengthening by means of soil nailing, anchorage, lateral resistant piles or structural cut-off wall can be an economical solution. These options have been used individually or in combination with good success as illustrated in the case histories presented.

All the above options explored are mostly for soil creep problem associated with gravitational load where the influence zone is limited, particularly in the vertical dimension. No many practical solution seems to be devised for tectonic movement though option of spanning across a fault movement can be tackled by flexible supports to the structure. It is still advisable to avoid constructing important structure over the tectonically active ground.

However, Tsai & Meymand (2013) presented an application of using segment vault to protect underground pipeline subjected to fault offset (Hayward Fault) for the upgrade of the Bay Division Pipelines (BDPLs) by San Francisco Public Utilities Commission. The alignment of Nos. 3 and 4 pipelines crosses the Hayward Fault and has to be upgraded for potential seismic event. To protect the pipeline at the Hayward Fault crossing, BDPL No. 3X, a new pipeline that will be installed parallel to Nos. 3 and 4, will be enclosed in a segmental, reinforced concrete vault with special joints that can accommodate lateral offset and compressive deformation during fault rupture. This design as shown in Figure 17 will allow rotation and compression of the pipeline at the ball and slip joints respectively. The simulated vault deformation conformed to design assumptions to build rigid concrete segments with relatively flexible joints oriented at 45° with respect to the vault longitudinal axis such that the fault deformation accommodated the relative slip and rotation between the joints.

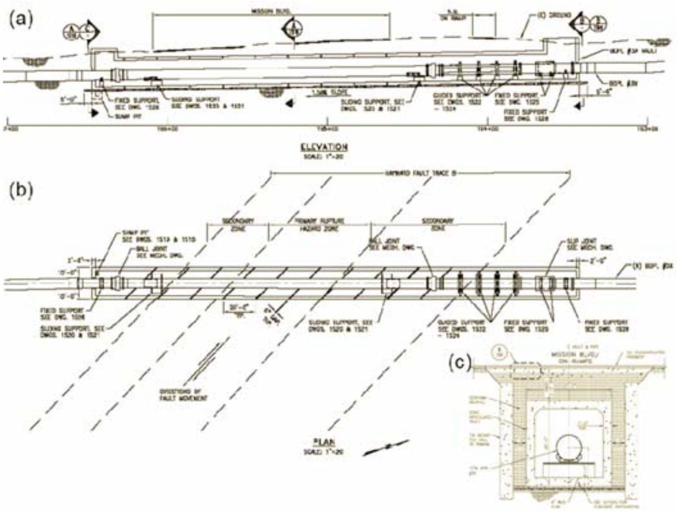

Figure 17: Design solution of BDPL 3X at the Hayward Fault crossing, (a) Cross section along the pipeline longitudinal direction, (b) Plane view, (c) Cross section along the pipeline transverse direction.

6.0 RECOMMENDATIONS

Based on the few explored engineering solutions, they all share one common pre-requisite for its success, i.e. to identify the creeping condition. Therefore, it is important that mapping work and studies to be carried out to classify the area based on its sensitivity condition. These areas can be classified into:

a. "stable formation" where normal structure with the standard design consideration can be applied,
b. "less sensitive" area where further investigation required with suitable treatment and types of structure that can be constructed and
c. "critical" area where permanent structure might not be suitable or required further investigation and study.

With such mapping and information, it will be able to facilitate the planning of the infrastructure development with better understanding of the ground condition and the suitable engineering solutions.

For design planning and construction on potential creeping ground, the following recommendations are made.

a. It is advisable to adopt critical state strength for slope stability design to minimise the risk of creep failure except for remediating failed slope without removing collapsed debris, in which residual strength shall be considered.
b. Inclinometer is a very useful instrumentation tool to reveal the ground deformation with time to provide creep behaviour observation. It is advisable to have inclinometer monitoring to confirm and detect the potential location of shear surface if the ground is expected to creep.
c. Loose fills have high tendency for creep movement when subjecting to load imposition or stress relief by toe erosion or excavation. Hence, proper compaction of placed fill will prevent failure within the fill.
d. Soft compressible materials shall always ne expected at the valley. Removal of such soft deposit and site clearing shall be religiously executed.
e. Natural valley is natural channel for accumulation of groundwater and also infiltrated water from the fill placed over it. Proper subsoil drainage is the crucial design element to control fluctuation of moisture within the fill.
f. Proper subsoil drainage shall always be provided at natural valley before fill placement.
g. Erosion failure at the downstream of drain discharge is also common if the collected surface runoff and subsoil drain discharge are not properly channelled to the natural stream. Necessary energy dissipation design detailing shall be considered if needed. Leaving the drainage discharge at the interface of the right of way (ROW) will not have the problem solved. When erosion occurred and detected, the remedial cost can be tremendous as most of the erosion failure occurs at fill embankment over the natural valley, where usually the drainage discharge will be located.
h. Where possible and practical, adverse geological features shall be mapped by an experienced engineering geologist for determination of potential geo-hazards and assessment of creep failure. However, it is extremely difficult to explicitly identify relict geological structures in completed weathered soils. But if the adverse geological structures can be detected at the nearby outcrops, it is reasonable expect that the same configuration of relict structures may exist in the weathered residual soils, which is derived from the same parent formation.

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