

RECENT ADVANCES IN PERMANENT GEOSYNTHETIC-REINFORCED SOIL STRUCTURES IN JAPAN

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Abstract: A number of geosynthetic-reinforced soil (GRS) retaining walls with a full-height rigid facing have been constructed by the stage construction method for railways and highways in Japan for the two decades. The construction and long-term behaviour of a GRS bridge abutment and a preloaded and prestressed GRS bridge pier, both directly supporting railway bridge girders, is described. Results from a comprehensive series of model loading tests performed to develop these technologies as well as field full-scale loading tests are reported. Moreover, a bridge abutment consisting of the backfill of geogrid-reinforced cement-mixed gravel and a thin RC facing structure constructed more recently to support a bridge girder for a new bullet train is described. This structure is unique in that the backfill supports laterally the RC facing, rather than exerting static and dynamic earth pressure on the facing. Results from triaxial compression tests on cement-mixed gravel and dynamic loading model tests as well as field full-scale loading tests are also reported.

INTRODUCTION

Geosynthetics, including polymer geogrids, have been increasingly used to reinforce the backfill of permanent soil structures, such as soil retaining walls (RWs) and bridge abutments (Tatsuoka & Leshchinsky 1994; Tatsuoka et al. 1997a; Ling et al. 2003). Some geosynthetic-reinforced soil (GRS) RWs of this type that were located in the seriously affected area of the 1995 Hyogo-ken Nambu Earthquake (i.e., the 1995 Kobe Earthquake) performed very well, showing their very high seismic stability (Tatsuoka et al. 1996a&b, 1998). Moreover, a number of GRS bridge abutments directly supporting bridge girders have been constructed. The most unique GRS structure is perhaps a GRS bridge pier that was constructed introducing vertical preloading and prestressing to the reinforced backfill aiming at substantially decreasing the residual deformation and increasing the seismic stability (Tatsuoka et al. 1997b; Uchimura et al. 1996, 1998a&b, 2001b, 2003b&c). In parallel to the actual construction projects as described above, a comprehensive series of model static and dynamic loading tests as well as field full-scale loading tests were performed to develop this technology (Shinoda et al. 1999, 2003a&b, Uchimura et al. 2001a, 2002, 2003a; Nakarai et al., 2002). This paper firstly briefly summarizes recent developments of this technology.

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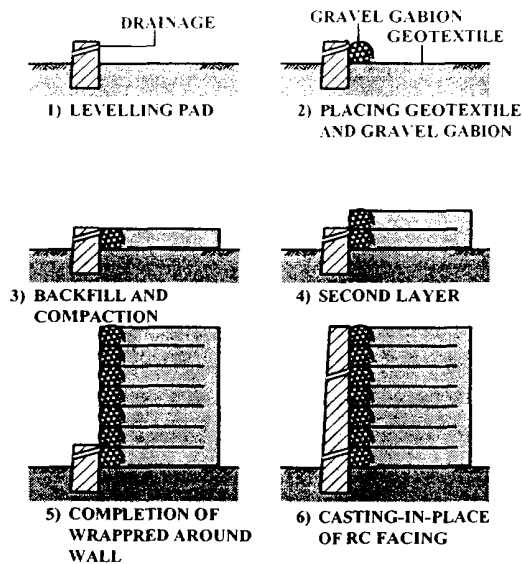


Fig. 1 Standard staged construction procedures for GRS-RWs having a full-height rigid facing (Tatsuoka et al., 1997a).

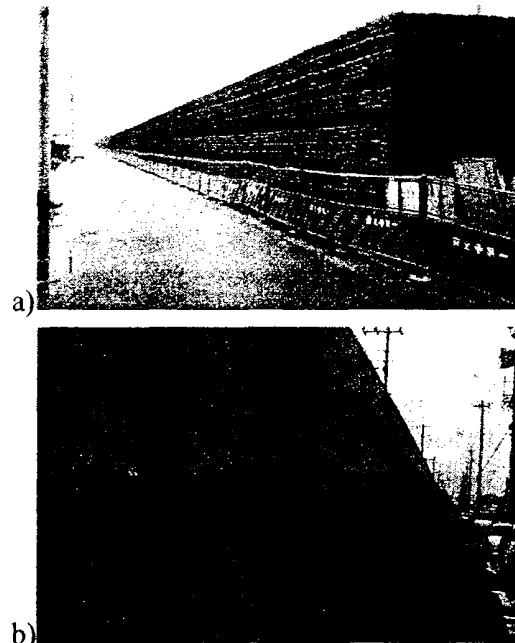


Fig. 2 A GRS-RW; a) under construction; & b) completed (Nagoya).

The second topic is the recent construction (2002 - 2003) of a bridge abutment consisting of the backfill of geogrid-reinforced cement-mixed gravel and a thin RC facing structure (i.e., a parapet) supporting a bridge girder for a new bullet train in Kyushu, Japan. The parapet is fixed to the cement-mixed gravel backfill by using a number of geosynthetic reinforcement layers. This structure is unique in that the backfill supports laterally the RC facing, rather than exerting static and dynamic earth pressure on the facing. Results from triaxial compression tests on cement-mixed gravel, which were referred to when determining the mixing proportion of the material and the water content when mixed and compacted, as well as results from dynamic loading model tests and field full-scale lateral loading tests of the completed structure, which were performed to ensure the connection strength between the parapet and the backfill, are also reported.

GEOSYNTHETIC-REINFORCED SOIL RETAINING WALLS

General

The construction method of permanent GRS-RWs that has been developed by the author and his colleagues is illustrated in **Fig. 1**. This construction method has become one of the standard RW construction methods in Japan. It is characterized by the following features:

- 1) The use of a full-height rigid facing that is cast-in-placed by the following staged construction procedure:
 - a) A leveling pad for the facing that is constructed at a later stage is first constructed.

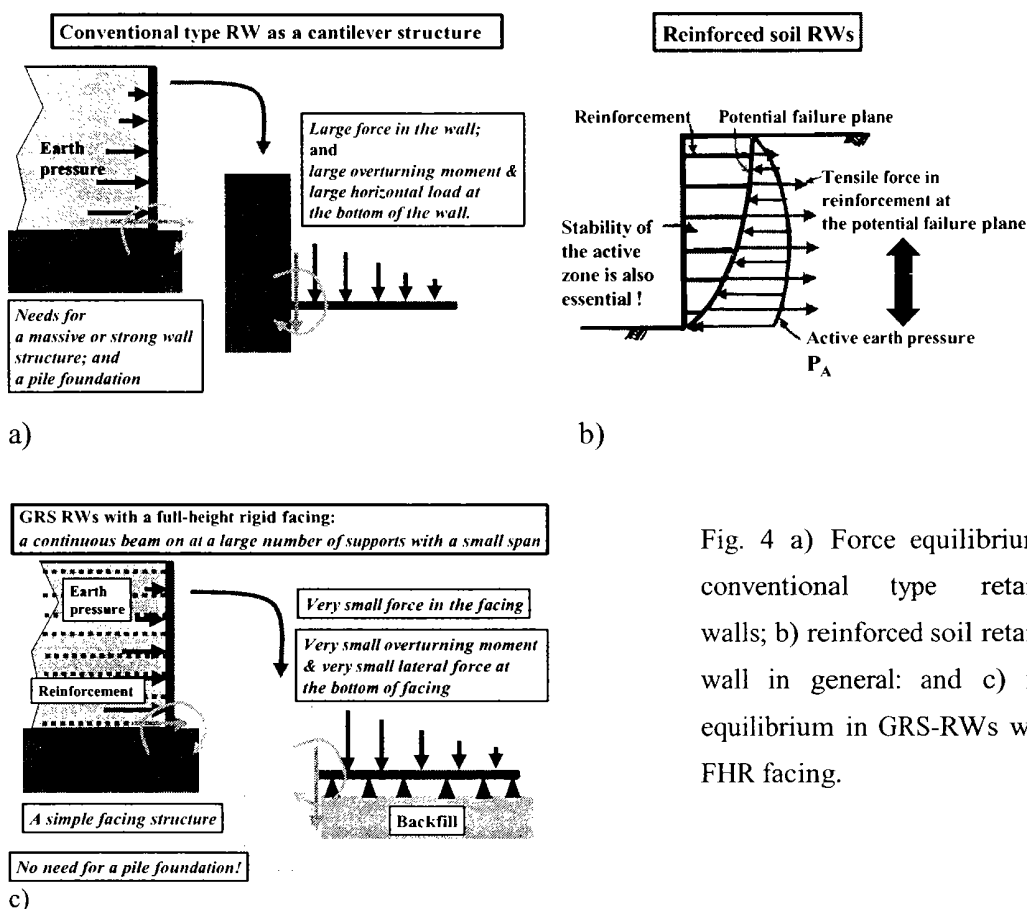


Fig. 4 a) Force equilibrium in conventional type retaining walls; b) reinforced soil retaining wall in general; and c) force equilibrium in GRS-RWs with a FHR facing.

reinforced soil RWs are generally flexible enough.

- 3) The performance of reinforced soil RWs could be equivalent to, or even better than, conventional type RWs.

These characteristic features are based on the following basic mechanisms.

First, a conventional type RW is usually designed as a cantilever structure resisting at its base against the active earth pressure exerted from the unreinforced backfill (**Fig. 4a**). For this reason, the internal moment and shear force activating in the facing structure and the overturning moment and sliding force developing at the bottom of the wall structure increase at a large rate with an increase in the wall height. On the other hand, with reinforced soil RWs, the backfill is retained by tensile force in the reinforcement (**Fig. 4b**). For this reason, the overturning moment and sliding force developing at the bottom of the wall is generally small.

Second, when the facing has a degree of flexural rigidity, the earth pressure acting on the back of facing does not become very small unless the backfill soil has a large cohesion intersect, or unless a large soil arching develops between two vertically adjacent reinforcement layers. If the earth pressure activated on the back of facing is essentially zero, the tensile force at the connection between the reinforcement and the back of facing also

becomes essentially zero, which results into a largely reduced soil-retaining capability of reinforcement (Tatsuoka 1993). Moreover, as the earth pressure decreases, the lateral confining pressure activating on the active zone of backfill decreases, and then the active zone becomes more deformable and less stable, particularly when the backfill is cohesionless. For this reason, sufficiently high connection strength between the reinforcement and the facing is essential.

Third, the use of a full-height rigid facing is very effective in increasing the wall stability and reducing the wall deformation, making reinforced RWs much more stable and more stiff when compared when using a relatively flexible facing such as a discrete panel facing or a wrapped-around facing. In the current design method for GRS-RW system, a full-height rigid facing is designed to support the active earth pressure that would develop in an unreinforced backfill (Horii et al. 1994). Despite the full mobilization of active earth pressure, the internal moment and shear force working in the facing and the overturning moment and sliding force acting at the bottom of facing could be very small, because a FHR facing behaves as a continuous beam supported by a number of reinforcement layers with a very short vertical spacing (i.e., 30 cm) (**Fig. 4c**). Therefore, the facing could be very thin and the required amount of steel-reinforcement in the facing could be a minimal. The minimum facing thickness specified for the GRS-RW system is 30 cm, which is based on workability considerations for concrete placing. This thickness is usually much larger than the one determined based on structural requirements. In addition, a pile foundation to support the facing is usually not necessary, mainly because the wall behaves as a self-supported structure (**Fig. 4c**).

Advantages of staged construction method

The facing for reinforced soil RWs should be flexible and deformable enough during construction to accommodate some large deformation of the backfill and underlying subsoil layer. On the other hand, completed reinforced soil RWs should be rigid enough with minimum residual deformations for its high serviceability. This contradiction can be resolved by the staged construction method for a FHR facing (**Fig. 1**), which has the following advantages:

- 1) Potential damage to the connections between the facing and the reinforcements due to settlements of the backfill relative to the rigid facing could be avoided.
- 2) A good compaction of backfill adjacent to the back of the facing could be achieved while allowing relatively large outward lateral displacements to occur at the temporary wall face. Accordingly, sufficiently large tensile strains could be developed in the reinforcement.
- 3) As the facing is constructed after the major part of the potential deformation of backfill and subsoil layer has taken place, a high alignment of facing can be ensured. Furthermore, the facing becomes free from the effects of vertical downward force caused by the

settlement of backfill relative to the facing during and after the compaction of backfill.

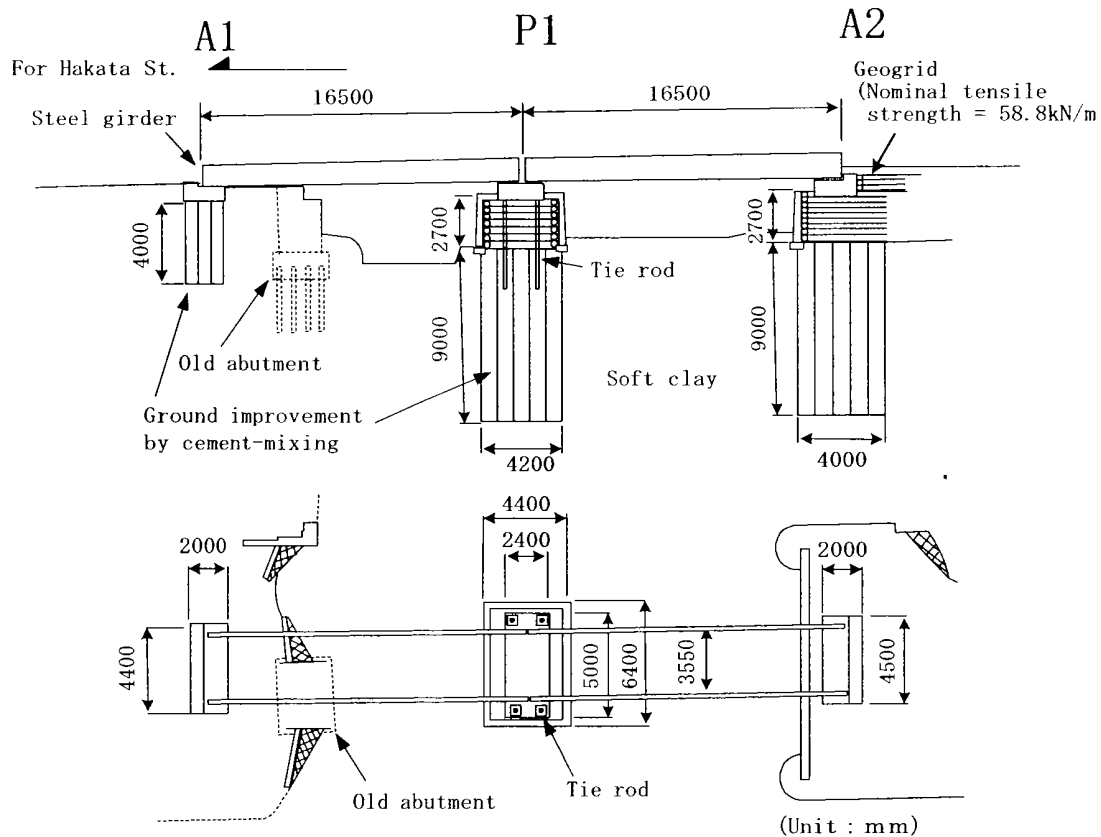


Fig. 5 Maidashi Bridge with PLPS reinforced soil pier.

PRELOADED AND PRESTRESSED GRS BRIDGE PIER

Preloading and prestressing procedure

The preloaded and prestressed (PLPS) reinforced soil method (Tatsuoka et al. 1997a&b) was developed to make reinforced backfill very stiff and essentially elastic while very stable against vertical and lateral seismic load. As the first prototype PLPS GRS structure, a bridge pier was constructed in the summer of 1996 to support two 16.5 m-long steel bridge girders for a single railway track in Fukuoka City, Kyushu, Japan (pier P1 in **Fig. 5**). Referring to Fig. 5, the typical construction procedures are described below:

- 1) The sub-soil is improved, if necessary, usually by cement-mixing-in-place.
- 2) A rigid bottom reaction block is constructed on the ground, to which four steel tie rods are vertically installed.

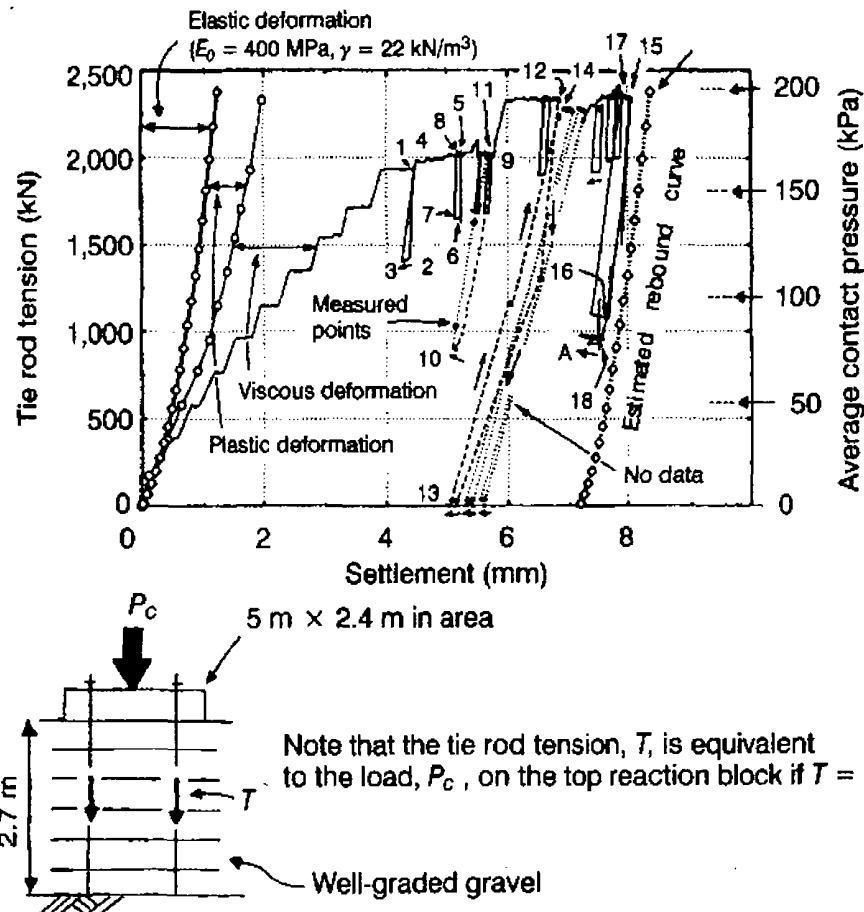


Fig. 6 Deformation of the backfill of pier during PL & PS procedure and subsequent period.

- 3) The backfill is constructed, being reinforced with geogrid reinforcement while holding the tie rods inside. The use of well-graded gravel is preferred for the backfill and a high degree of compaction of the backfill is essential.
- 4) A top RC reaction block is constructed on the top of the completed backfill.
- 5) A set of hydraulic jacks is arranged at the top ends of the tie rods, supported by the top reaction block.
- 6) The backfill is vertically preloaded by using the jacks, developing as large as possible compression of the backfill by keeping sufficiently high constant preload for a long period or applying many times of cyclic loads.
- 7) The vertical load is partially released to a prescribed non-zero prestress level.
- 8) The top ends of the tie rods are fixed to the top reaction block, usually by using nuts, before removing the jacks. A more sophisticated device, "a ratchet connection system" described later, could be used in place of nuts to maintain better the prestress for a long duration.
- 9) The vertical stress remaining in the backfill, which is in equilibrium with the tie rod tension, works as the prestress.

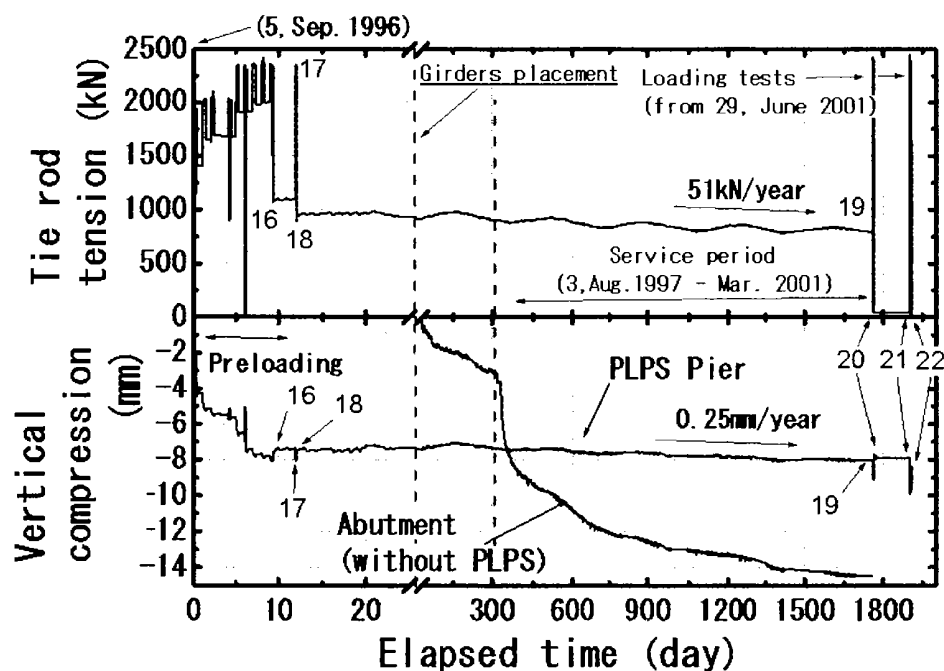


Fig. 7. Long-term time histories of total tie rod tension of pier and vertical compressions of pier & abutment.

The mechanisms and associated benefits of the PLPS method could be summarized as follows:

- The preload that could be applied to the reinforced backfill without damaging is much higher than when the backfill is not reinforced.
- The amount of unloading from the preload level to the initial prestress level should be larger than the maximum design load that would be applied during service so that the deformation of the backfill against external vertical loads could be essentially elastic. On the other hand, the prestress should not be made very low or zero to avoid swelling and associated softening of the backfill (Shinoda et al., 1999, 2003a).
- The reinforced backfill structure that is being subjected to sufficiently high prestress has a high stiffness for small strains as operated at working loads, thus a high structural integrity, and therefore can become very stiff also against monotonic and cyclic lateral load (Uchimura et al. 2001a, 2002).
- The vertical load working on the top of the backfill is always in equilibrium with the sum of the external vertical load applied on the top reaction block and the tie rod tension. When external vertical compressive load is applied at the top of the structure and the backfill is compressed accordingly, the tie rod tension decreases associated with vertical compression of the backfill. This behaviour results in a reduction of the vertical load that is to be applied to the backfill. This mechanism decreases the compression of the backfill for the same amount of external load.
- Large part of tensile strains in the reinforcement that have developed by preloading could

remain after the vertical load is decreased to the prestress level. Therefore, the reinforcement could confine the backfill deformation more efficiently than when the PLPS procedure is not used.

- f) When the backfill is preloaded at a sufficiently high level for a long period, relatively large creep deformation would take place in the backfill. Then the rate of relaxation of tie rod tension under the prestressed condition could become very low for a long period.
- g) The backfill may deform both in a bending mode and in a simple shear mode during a seismic event. If the backfill is very well compacted, large dilatation of the backfill could develop associated with large simple shear deformation. Such bending and shear modes of deformation could be

effectively restrained by vertically arranging tie rods, resulting in a considerable increase in the tie rod tension and pressure levels in the backfill during seismic events (Uchimura et al., 2001a, 2002, 2003a). Therefore, a high seismic stability can be expected.

Construction of a prototype PLPS GRS bridge pier and abutment

For the pier shown in Fig. 5, the design dead load by the girder weight and live load by train including impact load are 196 kN and 1,280 kN. The bridge was used for a period from August 1997 until the end of March 2001. Subsequently, full-scale loading tests were performed on the pier and a geogrid-reinforced soil abutment for the same bridge (A2 in Fig. 5). A well-graded gravel of crushed sandstone ($D_{max} = 30$ mm, $D_{50} = 0.9$ mm and $U_c = 16.5$) was well compacted and a geogrid reinforcement of polyvinyl alcohol coated with polyvinyl chloride (PVC) was used. The nominal rupture strength was 73.5 kN/m and the nominal stiffness was 1,050 kN/m at strains less than 1 percent. The average vertical spacing of the reinforcement layers was equal to 15 cm. Abutment A2 was a conventional type geogrid-reinforced soil retaining wall (GRS-RW), which was constructed by the same method as the pier, except that it had only one wall face and the preloading & prestressing procedure was not applied. The backfill was reinforced with the same type of geogrid as used for the pier

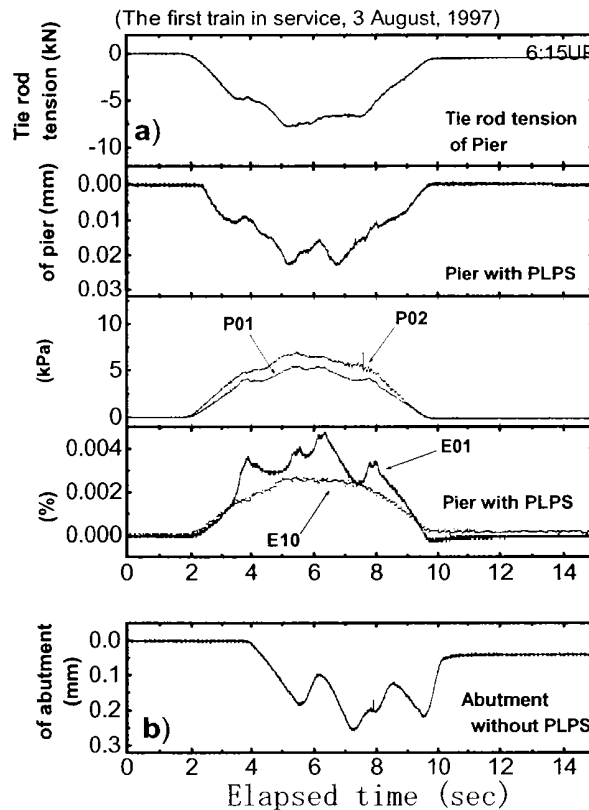


Fig. 8. Behaviours of a) the pier and b) the abutment during the first train passing in service (6:15 AM, 3 August, 1997).

at a vertical spacing of 30 cm. Both sides of the backfill consisted of exposed slopes (1.5:1.0 in H:V) without a facing.

The preloading work started ten days after casting-in-place the top reaction RC block (5 m-long, 2.4 m-wide and 0.8 m-thick). **Fig. 6** shows the relationship between the total tie rod tension, T , and the backfill compression during the PL & PS procedure. At the PL stage, the tie rod tension was the same as the total vertical load working on the crest of the backfill. The rebound and re-compression of the backfill that took place during a cycle of unloading and reloading with a load amplitude of 1,400 kN applied in the last day (between stages 17 and 18) were nearly the same and equal to only 0.4 mm. This small deformation already indicated a very high stiffness and nearly elastic deformation characteristics of the preloaded and prestressed backfill. The elastic deformation presented in Fig. 6 was obtained from this measured unload-reload behaviour while referring to results from triaxial tests on the backfill (Uchimura et al. 2003b&c). The time-independent inelastic (or plastic) component of the compression was the difference between the integrated instantaneous compression during the primary loading and the elastic component. The difference between the total compression and the instantaneous component (elastic plus plastic components) is the time-dependent (viscous) component. The viscous component was more than a half of the total compression and its ratio to the total compression increased with the load level. This result indicates the paramount importance of removing potential creep deformation of the backfill by sufficiently large preloading. Note that theoretically the plastic and viscous components cannot be separated from each other, but they should be treated in total as visco-plastic strains (or irreversible strains).

It may also be seen from Fig. 6 that the average stiffness of the backfill when reloaded from Point 10 was noticeably smaller than the value when reloaded from a higher load level (e.g., from Point 7). Moreover, the average stiffness when reloaded from zero load (from Point 13) was smaller than the above. This behavior was likely due to the effects of swelling and associated softening of the backfill when unloaded to zero. This result shows the importance of maintaining sufficiently high prestress in the backfill during service.

Subsequent to the PL & PS procedure, 30 cm-thick full-height rigid facings of unreinforced concrete were cast-in-placed on the four wall faces. The total construction period was about 1.5 months. The construction cost for the pier was about a half of the one for an equivalent conventional RC pier supported by a pile foundation.

Long-term behavior

After the unreinforced concrete facings were cast-in-placed, two steel bridge girders, each weighing 211 kN, were installed on the pier and the abutments. The instantaneous

compression of the pier by a girder weight of 211 kN was only 0.04 mm, while that of the abutment by a girder weight of 105 kN was ten times larger, equal to 0.4 mm. Then, the behavior of the pier and the abutment was monitored for more than five years by the end of project (Fig. 7).

It may be seen from Fig. 7 that the tie rod tension and the compression of pier P1 changed only slightly for this long period, which was due mostly to be annual changes due to temperature effects. The tensile strains in the reinforcement were kept almost constant without showing any creep extension. On the other hand, abutment A2 showed noticeable creep compression by the self-weight and the girder weight. The total compression of the abutment after the girder was installed was 3.1 mm by the summer of 1997. The difference shows the first advantage of PL & PS that makes long-term creep deformation of the backfill under static load condition very small. On 19 July 1997, a diesel locomotive of 637 kN in weight passed 6 times over the bridge for inspection. The residual compression of pier P1 was 0.02 mm, while that of abutment A2 was 0.52 mm.

The bridge has been opened to service 3rd August 1997. Fig. 8 shows the behavior of pier P1 and abutment A2 at the first train passing in service. The train consisted of 2 coaches, each

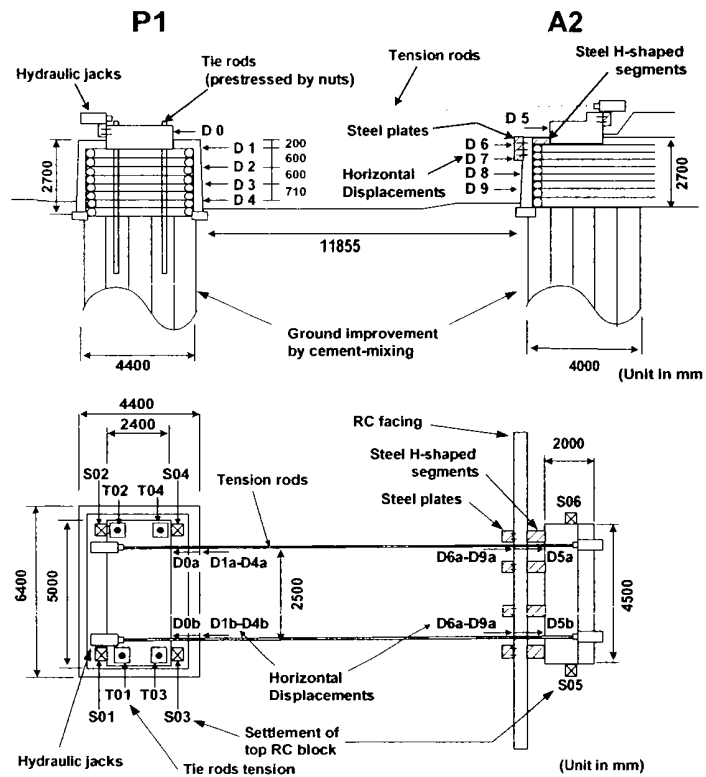


Fig. 9 Set up of horizontal loading tests performed in 2001.

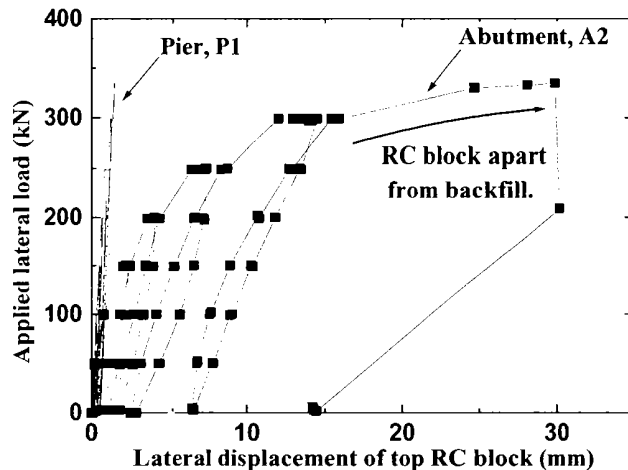


Fig. 10 Relationships between applied lateral load and lateral displacement of the top RC blocks of pier P1 and abutment A2.

weighing 353 kN excluding the weight of passengers. The amplitude of compression of the pier was 0.02 mm, which was equivalent to a vertical strain of 0.001 percent; this very small strain also suggests nearly elastic behavior of the backfill of the pier. The tie rod tension responded to the pier compression. The change in the tensile strain in the reinforcement was very small and elastic, which was due to very small compression of the backfill. Apparently, the backfill having highly elastic properties with high stiffness actually supported the trainload. These prominent backfill properties were achieved by sufficiently large preloading and prestressing, which was possible only with a help of reinforcement. On the other hand, the amplitude of the compression of the backfill of the abutment was 0.2 mm, which was not a harmful quantity, but 10 times larger than that of the pier. This difference shows the second advantage of preloading and prestressing that makes the amplitude of transient compression by dynamic loading very small. The rate of compression of the backfill of the pier after having been opened to service was very small, only 0.25 mm/year on average. Corresponding to the above, the decreasing rate of the tie rod tension was also very small, 51 kN/year. Shinoda et al. (1999, 2003b) suggested that the use of a ratchet connection system to fix the top ends of the tie rods to the top reaction block is very effective to keep the prestress sufficiently high even when relatively large compression takes place in the backfill during a long life time of the structure.

Creep deformation

It is true that geosynthetic reinforcement is more deformable and may exhibit larger creep deformation than metallic reinforcement. However, based on results from laboratory load/stress-strain tests of geosynthetic and backfill, behaviours of small-scaled reinforced soil models and proto-type full-scale geosynthetic-reinforced soil structures and associated numerical simulations of experimental results, including FE simulations, Tatsuoka et al. (2004), Hirakawa et al. (2004) and Kongkitkul et al. (2004) showed the following:

1. Creep is not a degrading phenomenon with both geosynthetic and backfill. The basic parameter to describe the stress/load-strain-time behaviour of geosynthetic reinforcement, backfill and reinforced soil is the strain rate (more rigorously, the irreversible or inelastic or visco-plastic strain). Therefore, the isochronous concept, for which time is the basic parameter, is not able to relevantly predict the load/stress-strain time behaviour of geosynthetic as well as soil and reinforced backfill subjected to arbitrary loading histories.
2. A non-linear three-component rheology model is relevant to represent the viscous properties of geosynthetic and backfill.
3. Tensile force in the geosynthetic arranged in the backfill subjected to sustained loading could decrease with time in ordinary full-scale cases, where the safety factor against the ultimate failure of structure under static loading conditions is sufficiently high.
4. The design rupture strength of geosynthetic obtained using a relatively larger creep reduction factor determined based on the conventional creep rupture curve could be overly

conservative.

4. By applying a relevant preloading history, creep deformation rate substantially decreases; therefore the possibility of creep rupture of geosynthetic reinforcement can be eliminated.

In summary, the excessive creep deformation and creep rupture is not an actual problem with usual full-scale geosynthetic-reinforced soil structures. In fact, no case having this kind of problem has been reported with geosynthetic-reinforced soil retaining walls constructed by the method illustrated in Fig. 1.

Full-scale lateral loading tests

Lateral loading tests of the pier and abutment were performed in June 2001. The top RC blocks of the pier and abutment were connected to each other with steel tension rods, and hydraulic jacks were set to pull simultaneously the top RC blocks (Fig. 9). H-shaped steel plates were arranged between the top RC block and the RC wall facing of abutment A2 to increase the lateral bearing capacity of the top RC by transmitting some part of the horizontal load to the top part of the RC wall facing. The facing was strengthened by bolting steel plates tightly to the facing before the lateral loading test. The top RC block and the RC facings of pier P1 were mechanically separated with a soft material arranged around the top RC block, thus almost all the horizontal load applied to the top RC reaction block was transmitted to the top surface of the backfill. Seven cycles of loading and unloading were applied while their peak load level was increased by 50 kN step-by-step up to 350 kN. The peak load was maintained for ten minutes in each

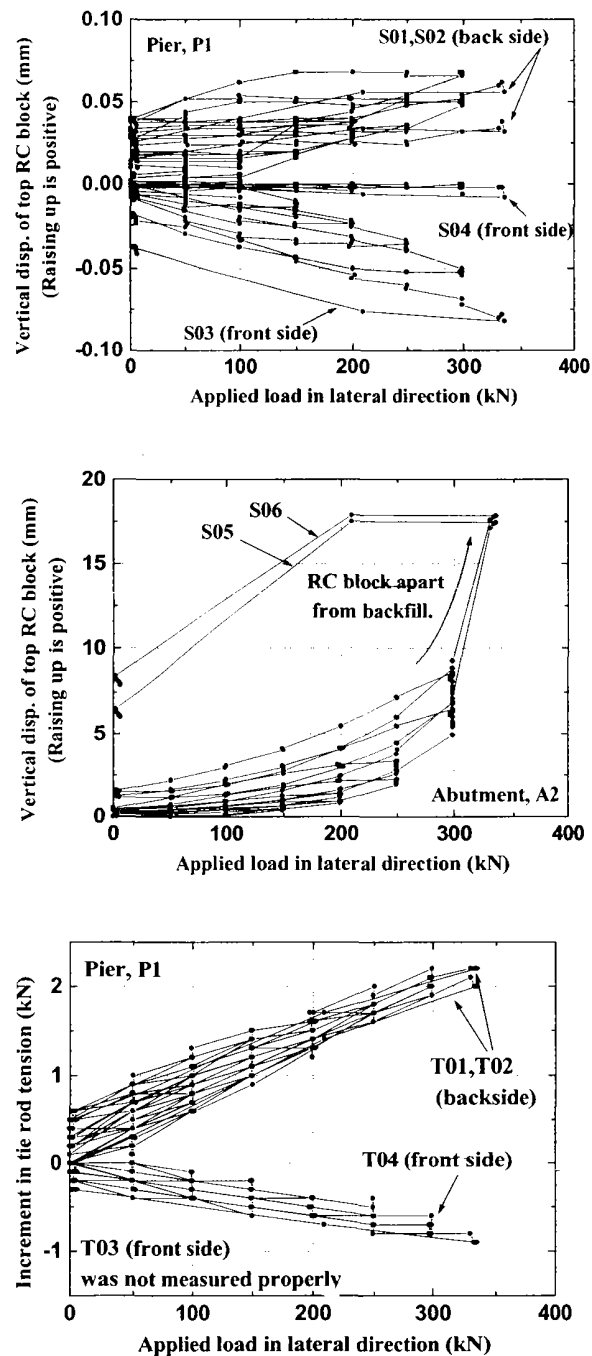


Fig. 11 Relationships between the applied horizontal load and the vertical displacement of the top RC blocks of a) pier P1; & b) abutment A2;

cycle, except the last cycles, in which abutment A2 failed.

Fig. 10 shows the horizontal displacements of the top RC blocks of pier P1 and abutment A2. The maximum displacement of the top RC block of pier was only 1.4 mm by horizontal load of 350 kN, while the abutment exhibited a displacement more than 20 times larger, and the top RC block turned over apart from the backfill. If the H-shaped steel plates had not been arranged between the top RC block and the RC wall facing, the abutment should have failed at a much lower load level. **Fig. 11a** shows the vertical displacements of the top RC block of the pier measured by using the displacement transducers shown in **Fig. 9**. It may be seen that the top reaction block of the pier exhibited very small overturning against larger horizontal load, which is due to the resistance of the tie rods against this mode of deformation of the pier (**Fig. 11c**). In comparison, as seen from **Fig. 11b**, the top reaction block of the abutment exhibited considerably larger overturning displacements.

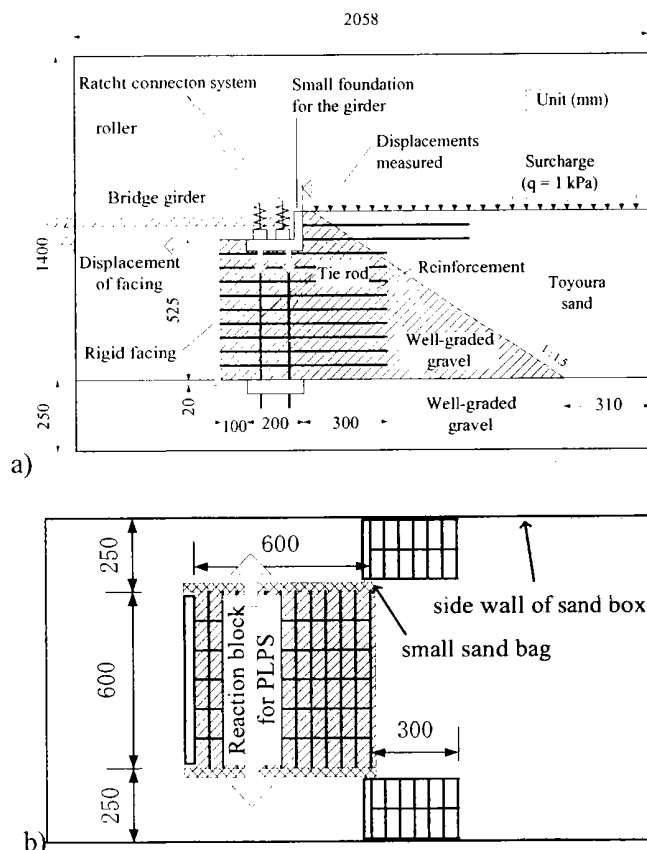


Fig. 12. GRS bridge abutment model.

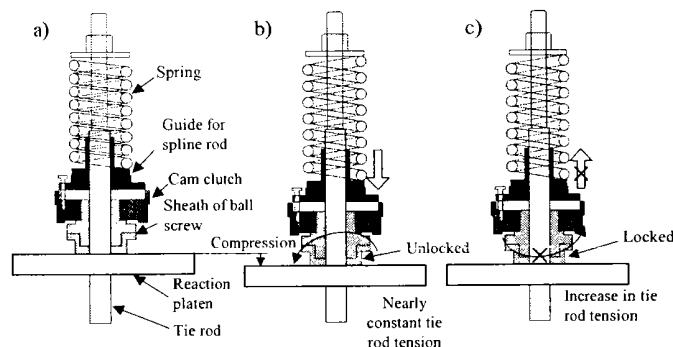


Fig. 13. Structure of the ratchet connection system.

The behaviour reported above shows that the preloaded and prestressed pier behaved as a nearly rigid mass, and the contact between the top RC block and the backfill was kept tight enough, supporting the horizontal load. This high performance was due to the high confining pressure associated with the vertical prestress applied on the top RC block. The prestress during the lateral loading tests were around 800 kN, which corresponds to an average confining pressure of 66 kPa on the base of the top RC block (cross-sectional area = 12 m²),

and 36 kPa on the backfill (cross-sectional area = 22 m² when not including the RC facings). For the maximum horizontal load applied in the tests was 350 kN, the mobilized friction angle on the horizontal surface in the backfill is $\arctan(350 \text{ kN}/800 \text{ kN}) = 23$ degrees, which is much smaller than the peak friction angle (equal to 60 degrees) of the backfill materials obtained from a triaxial tests on a specimen compacted to dry density of 1.95 g/cm³.

Summary

The comparison of the long-term behaviors of the preloaded and prestress pier and the abutment without preloading and prestressing showed that preloading and prestressing procedure can restrain: 1) creep compression under long-term static compressive load; 2) transient compression by dynamic loading; and 3) residual compression by dynamic load with a great number of loading cycles. The importance of keeping high prestress for high seismic stability was validated by full-scale lateral loading tests of the pier and abutment.

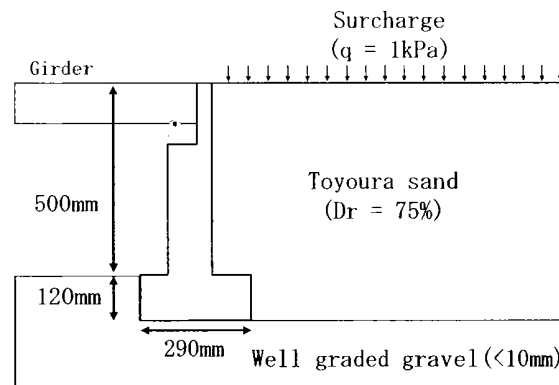


Fig. 14. Cantilever bridge abutment model.

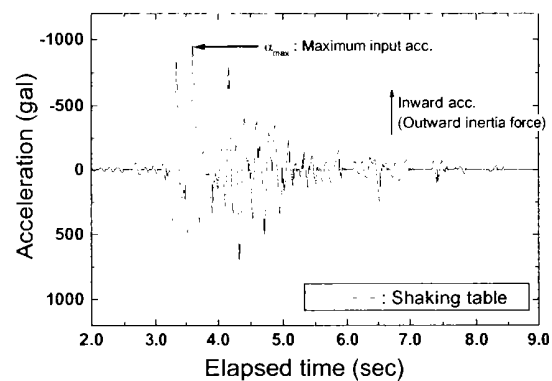


Fig. 15. Input acceleration wave from 1995 Hyogoken-Nambu Earthquake with a maximum acceleration of 1,000 gal.

SEISMIC STABILITY OF REINFORCED SOIL BRIDGE ABUTMENT

Introduction

The seismic stability of reinforced soil abutments made very stiff and stable by the preloading and prestressing procedure described in the precedent section was evaluated by performing shaking table tests on three-dimensional bridge abutment models. Effects of relaxation of the prestress, which would take place during long-term service under traffic load and by severe seismic loads, on the seismic stability were evaluated by changing the initial prestress level at the start of shaking. It was found that, as long as the expansion of backfill during shaking was prevented by relevant measures, a high seismic stability could be achieved even with a very small initial prestress.

Test method

A pair of models of GRS abutment and conventional RC abutment were constructed in a 110 cm-wide box on a shaking table (Fig. 12). The abutment and the backside embankment were constructed directly on a 250 mm-thick foundation layer of a well-graded gravel ($D_{max}=10$ mm, $\gamma_d=1.95$ gf/cm³). The model reinforced soil abutment consisted of a 60 cm-wide reinforced gravel backfill placed at the center of the box. The backfill was made by manually compacting air-dried well-graded gravel ($D_{max}=5$ mm; $U_c=11.3$; $\gamma_d=1.86$ gf/cm³), which was reinforced with 12 layers of 60 cm-long grid reinforcement at a vertical spacing of 50 mm. The grid was made of phosphor bronze strips (3.5 mm in width and 0.2 mm in thickness) having a surface made rough by gluing Toyoura sand particles. The aperture of the grid was 50 mm in the longitudinal direction and 100 mm in the transversal direction. The back face of the gravel backfill was inclined (1.0H : 1.5V) and the front face was covered with a full height rigid facing tightly connected to the reinforcement layers. The side faces with a length of 600 mm were protected by using flexible sandbags connected to the reinforcement layers. The backfill back of the reinforced gravel backfill was made by pluviating air-dried Toyoura sand ($D_{50}=0.2$ mm; $U_c=1.44$; $D_r=75\%$). A surcharge of 1 kPa was applied on the crest of the Toyoura sand backfill by using lead shots. Rigid reaction plates were set at the top and

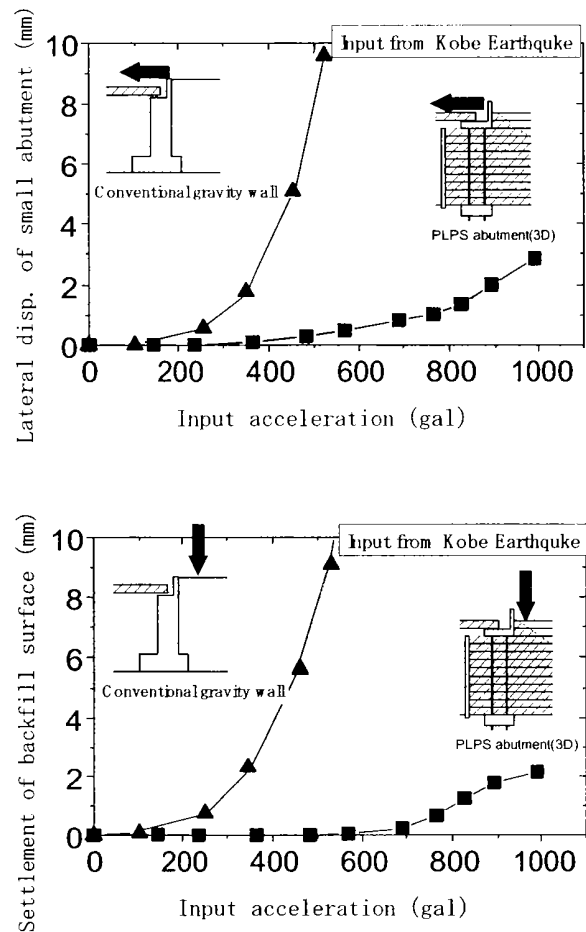


Fig. 16 Cumulative values of a) lateral displacement of the abutment; b) settlement at the top of the backfill.

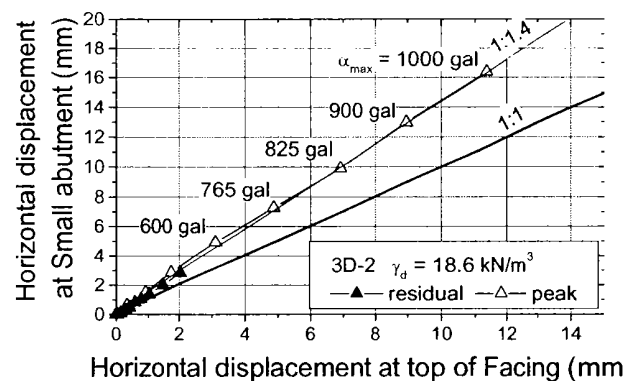


Fig. 17 Comparison of the lateral displacement of the top of the facing and the small foundation for the girder.

bottom end of the reinforced gravel backfill, to which the top ends of the four steel-wire tie rods were fixed. One end of a model bridge girder with a weight of 200 kg was fixed through a hinge to the top reaction plate. The model girder was also supported through a roller at the top of the end wall of the box (Fig. 12a).

The top ends of the tie rods were connected to the top reaction plate by using the “ratchet connection system” (Fig. 13a) having the following two prominent features: 1) Each tie rod is always pulled up by a spring (Fig. 13a). When the backfill is vertically compressed by any cause, the spring extends keeping the tie rod tension nearly constant (Fig. 13b). 2) When the backfill tends to vertically expand by volume increase due to dilatancy and/or bending deformation of the backfill associated with seismic loading, the cam clutch locks the compression of the spring, which makes the system very stiff (Fig. 13c), making the tie rod tension higher than the initial level. The reinforced gravel backfill was preloaded to an average vertical stress equal to 40 kPa and then unloaded to the initial prestress level equal to 20 kPa.

For a reference, a model of conventional type RC cantilever bridge abutment with a spread foundation, in place of a pile foundation, was prepared (Fig. 14). The backfill was air-dried Toyoura sand having $D_r = 75\%$. Fig. 15 shows the time history of the input acceleration, which was obtained by adjusting the time history of horizontal ground acceleration (NS component) recorded at the Marine Meteorological Observatory during the 1995 Kobe Earthquake. The time axis of the record was contracted so that the predominant frequency became 5 kHz to keep the ratio of the natural frequency of structure to the predominant frequency of input motion in the scaled-down model tests is essentially the same as the one in typical field full-scale cases. For each model, a series of shaking tests were performed by increasing step by step the peak acceleration, α_{max} , of the input motion from 100 gal to 1,000 gal with an interval of approximately 100 gals. With the reinforced soil abutment model, a shaking with a maximum acceleration of 1,000 gals was repeated while decreasing the initial prestress towards zero and finally removing the connection of the tie rods to the crest of the reinforced backfill.

Test results

Figs. 16a & b compare the residual lateral outward displacements and settlements at the crests of the reinforced soil and conventional cantilever abutments, which had accumulated by the end of respective shaking stage. The initial prestress of the reinforced soil abutment was 20 kPa at the start of the first stage shaking, which was not reset for the subsequent shaking stages. The following trends of behaviour may be seen from these figures:

- 1) With the cantilever abutment, the lateral displacement and settlement started increasing

already when $\alpha_{\max} = 200$ gals. At the subsequent shaking stages, the increasing rate of displacement increased with an increase in $\alpha_{\max}$. This catastrophic mode of failure was due to a loss of bearing capacity in the supporting ground beneath the toe of the spread footing. This mode of failure could be prevented by supporting the abutment with a pile foundation, which could however substantially increase the construction cost.

- 2) With the reinforced soil abutment, the displacements were very small even when $\alpha_{\max}$ became as high as 600 gals and substantially smaller than those of the cantilever abutment. Moreover, the displacements did not exhibit a high increasing rate with an increase in $\alpha_{\max}$. Note that this very high seismic stability and ductility was attained without being supported by a pile foundation.

Fig. 17 compares the maximum lateral displacements relative to the shaking table during shaking observed at the top of the facing (at a height of 475 mm from the ground surface) and the sill beam supporting the model bridge girder (at a height of 665 mm). The ratio between the displacements at the two heights was kept nearly constant with an increase in the input motion. Moreover, the ratio of the displacements was about 1.4, which is the same as the ratio between the two heights from the ground surface, $665/475 = 1.4$. These facts mean that the reinforced backfill, together with the facing and the top reaction plate (as the sill beam for the bridge girder), behaved like a monolith exhibiting an overturning displacements about the bottom of the facing. Such a performance as described above became possible because of high prestress maintained by using the ratchet system.

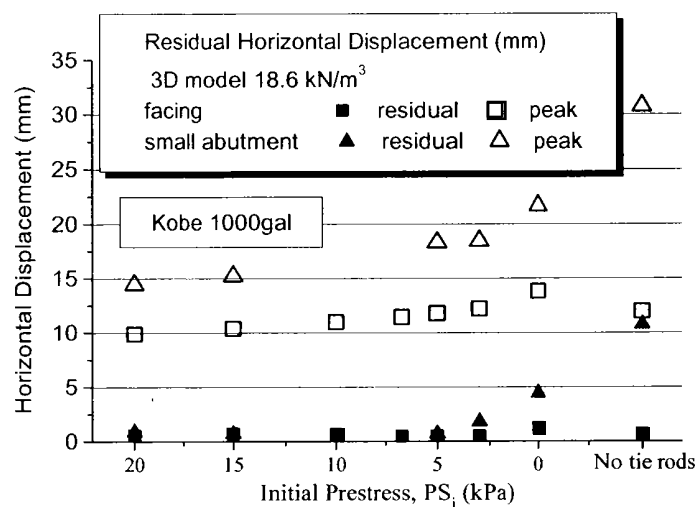


Fig. 18. Horizontal displacements at the top of the GRS-abutment model.

After the first series of shaking tests described above, the second series was performed on the reinforced soil abutment by repeating shaking with a maximum acceleration of 1,000 gal while decreasing stepwise the initial prestress from 20 kPa to 0 kPa. **Fig. 18** shows the increments of residual horizontal outward displacement and the peak horizontal outward displacement observed at the top reaction plate and the top of the facing at each stage of shaking. In these shaking tests, the ratchet connection system was not used only at the last

stage, denoted as “no tie rods”. At the stage denoted as “initial prestress equal to 0 kPa”, the top ends of the tie rods were fixed to the top reaction plate with zero initial prestress. Therefore, tension was activated in the tie rods when the height of the backfill became larger than the initial value. At the stage denoted as “no tie rods”, the top ends of the tie rods were not fixed to the top reaction plate with zero initial prestress. The following trends of behaviour may be seen from these figures:

- 1) The increments of residual horizontal displacement at each shaking stage were nearly constant until the initial prestress became 5 kPa. However, the peak horizontal displacement increased gradually with a decrease in the initial prestress before the initial prestress became 5 kPa.
- 2) The increments of residual horizontal displacement as well as the peak horizontal displacements gradually increased as the initial prestress became smaller than 5 kPa towards 0 kPa. Yet, the ratio between the peak lateral displacements at the top reaction plate and the top of the facing was kept to around 1.4, showing that the reinforced gravel abutment still behaved essentially like a monolith.
- 3) At the last stage without tie rods, both increment of residual horizontal displacement and peak horizontal displacement at the top reaction plate became much larger, while those at the top of the facing became slightly smaller, compared with the previous stages. At the same time, the top reaction plate exhibited a noticeable residual tilting. These facts indicate that the top reaction plate slipped at the interface between the top reaction plate and the top of the backfill during shaking. It seems that relatively large deformation of the reinforced gravel backfill and slipping at the interface between the top reaction plate and the top of the backfill took place when the prestress became nearly zero during shaking because of no use of tie rods.

Summary

The results from model shaking table tests presented above showed that preloaded and prestressed reinforced soil bridge abutments could have seismic stability and ductility that is substantially higher than conventional cantilever RC abutments without a pile foundation. The above could be achieved by keeping the prestress equal to, or larger than, the initial value and by preventing the expansion of reinforced backfill by fixing the top ends of tie rods to the reinforced soil abutment by using a ratchet connection system. The initial prestress when a ratchet system is used could be relatively low compared to the preload that is necessary to make the backfill elastic for design static and live loads that would be applied to the abutment during service. In that case, a certain amount of relaxation could be allowed in the prestress for a long service period as far as the prestress does not become nearly zero. It appears that a relatively simple maintenance system for tie rod tension that can satisfy the requirement described above can be designed.

BRIDGE ABUTMENT MADE OF CEMENT-MIXED GRAVEL BACKFILL

General

A great number of conventional type railway bridge abutments (**Fig. 19a**) were seriously damaged with a large relative settlement between the abutment and the backfill by a number of previous major earthquakes in Japan. Such a relative settlement (i.e., a bump) as above could endanger safe train operation even when it is small, say 10 cm. To prevent this type of damage, in 1978, the Japan National Railway (presently several Japan Railway Companies) introduced a new type of backfill (**Fig. 18b**): i.e., a triangle-shaped backfill zone, called “approach block”, is constructed immediately behind the abutment by well compacting a well-graded gravel. However, a number of this type of abutments did not perform satisfactorily during subsequent major earthquakes. Most recently, the approach blocks of well-graded gravel for a number of bridge abutments for new bullet train lines were improved by mixing with cement to minimise the settlement of the backfill relative to the RC abutment. However, as the approach block was not connected to the RC facing structure, the dimensions of the RC facing structure did not decrease substantially despite the use of the backfill of cement-mixed gravel.

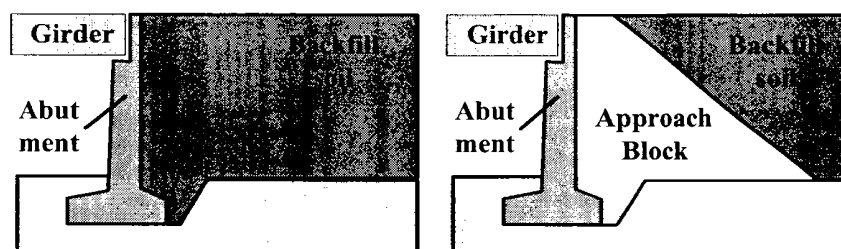


Fig. 19 Typical conventional bridge abutment types; a) most conventional (model 1 in the shaking table test); and b) structure introduced in 1978 (model 2 in the shaking table test) (Watanabe et al., 2002).

In view of the above, a long-term joint research project started 1997 aiming at developing new aseismic cost-effective types of bridge abutment, for which the author was one of the principal researchers. Eventually, a bridge abutment consisting of the backfill of geogrid-reinforced cement-mixed gravel and a thin RC facing structure (i.e., a parapet) was developed. The first prototype was recently constructed to support a bridge girder for a new bullet train at Takada in Kyushu (**Fig. 20a & b**). This new type is unique in that the backfill supports laterally the parapet, rather than exerting static and dynamic earth pressure on the parapet. The reinforcement layers are connected to the facing structure: a) to restrain the settlement of the approach block relative to the RC facing structure; and b) to maintain a high integrity of the abutment structure, in particularly under severe seismic loading conditions. By this connection, the RC facing structure becomes a continuous beam supported with a large number of sports with a small spacing and therefore it becomes substantially less

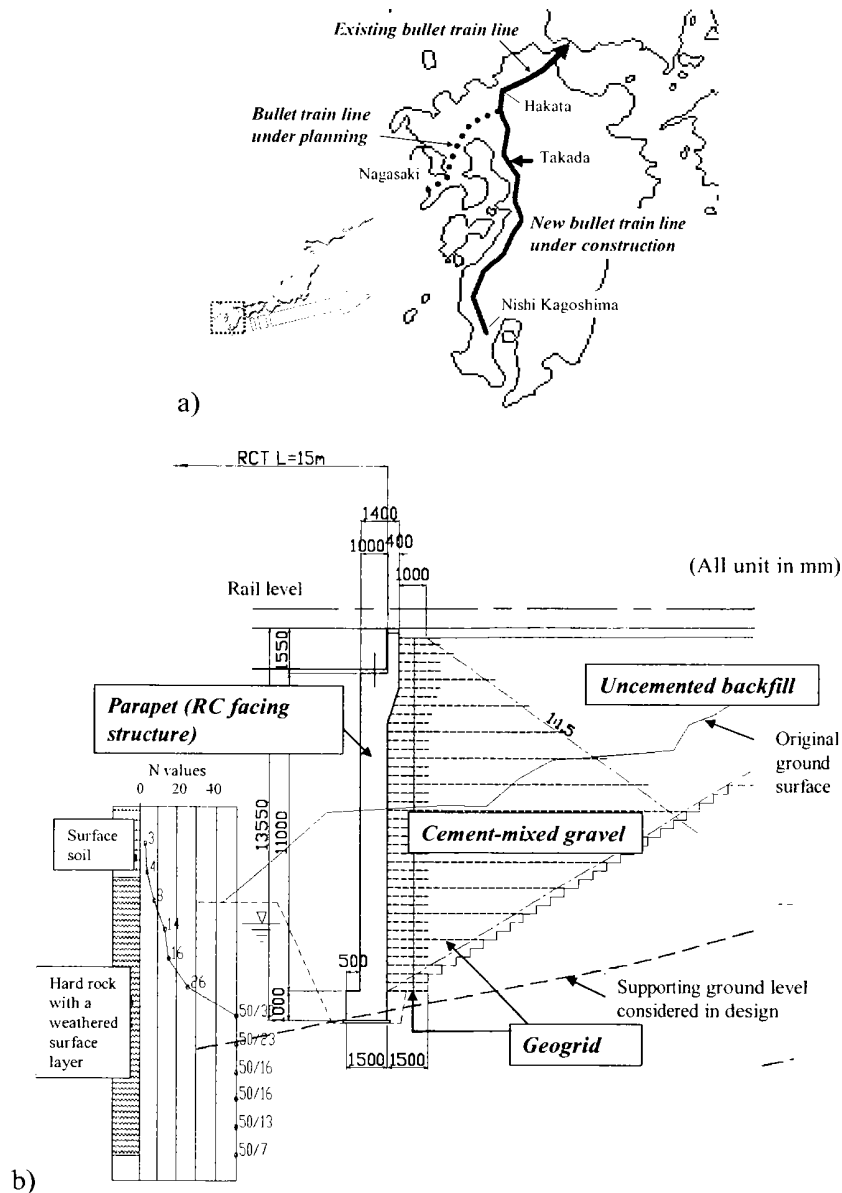


Fig. 20 a) Location; and b) structure of new type bridge abutment constructed at Takada (Aoki et al., 2002).

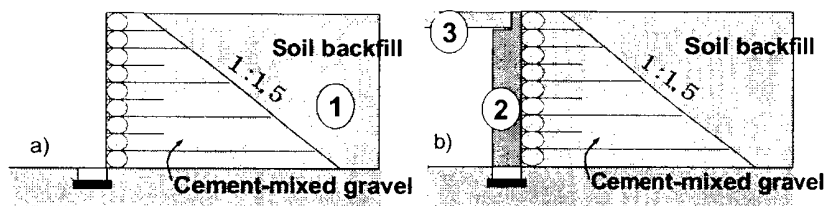


Fig. 21 Staged construction: the numbers show the construction sequence.

massive than the conventional type RC bridge abutment. Moreover, as the facing is constructed after the backfill is completed (**Fig. 21**), the connection between the facing and the backfill is not damaged by the relative settlement between them during construction. In addition, the stability of the complete structure is not controlled by the bearing capacity of

ground immediately below the facing, unlike the conventional type bridge abutments. Therefore, it is usually unnecessary to support the facing with piles. For these reasons, this new type could be much more cost-effective than the conventional type.

Shaking table tests

Table 1 Abutment models tested (Watanabe et al., 2002).

Model name	Backfill	Reinforcement	Input motion	Width of footing for RC facing
Model 1 (Conventional)	No approach block (Toyoura sand, $D_r = 75\%$)	No	Sinusoidal wave & Kobe wave	390 mm
Model 2 (Conventional)	With approach block of dry gravel ($\gamma_d = 1.9 \text{ g/cm}^3$)	No	Sinusoidal wave	390 mm
Model 3 (Conventional)	With approach block of cement-mixed gravel	No (backfill and facing unconnected)	Sinusoidal wave	390 mm
Model 4 (New type)	With approach block of cement-mixed soil	Yes (backfill and facing connected)	Kobe wave followed by sinusoidal wave	290 mm
Model 5 (New type)	With approach block of cement-mixed soil	Yes (backfill and facing connected)	Kobe wave followed by sinusoidal wave	200 mm

A series of model shaking table tests were performed to compare the seismic stability of the new type abutment (Figs. 20 & 21) with that of the conventional types (Fig. 19); and to ensure a high seismic stability of the new type during very high-intensity seismic load (so-called Level 2 earthquake) (Watanabe et al., 2002). The following five types of abutments were investigated (**Table 1**):

Model 1: the most conventional type with the backfill of uncemented soil (Fig. 19a).

Model 2: a conventional type with an approach block of well-compacted uncemented well-graded gravel (Fig. 19b).

Model 3: a conventional type with an approach block, as shown in Fig. 19b, that is cemented soil not reinforced with geogrid reinforcement (so the backfill and the facing are not connected to each other).

Models 4 & 5: a new type with an approach block of well-compacted cemented soil reinforced with reinforcement layers connected to the facing, having different widths of the facing footing (**Fig. 22**).

The facing was made of aluminium with a height of 620 mm having a footing base with a width of 390 mm (Models 1 – 3), 290 mm (Model 4) and 200 mm (Model 5). The facing supported a model bridge girder with a weight of 200 kg through a hinge support. So, full dynamic lateral load of the girder was activated to the facing. The unreinforced backfill of

Model 1 was made by air-pluviating air-dried fine sand (Toyoura sand) to $D_r = 75\%$. The approach block of Model 2 was a compacted well-graded gravel ($U_c = 10.7$; $D_{50} = 1.1$ mm, $D_{max} = 5.0$ mm and $FC = 5.2\%$; $w = 5\%$) with a dry density of 1.9 g/cm³. Despite that the actual approach block is a cement-mixed well-graded gravel, the model approach block of Models 3, 4 & 5 was a cement-mixed loam with $q_u = 200$ kN/m² while accounting for model similitude.

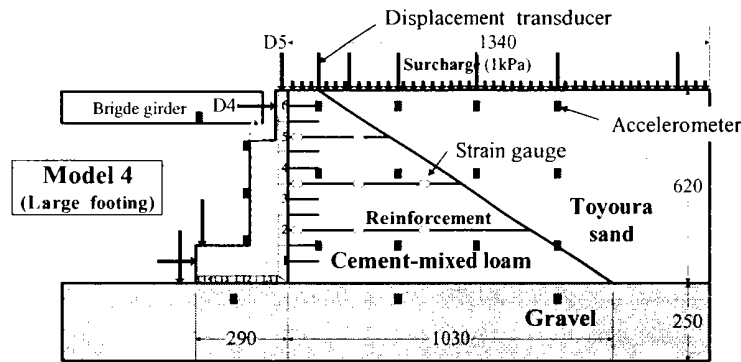


Fig. 22 Cross-sections of model 4 (Watanabe et al., 2002).

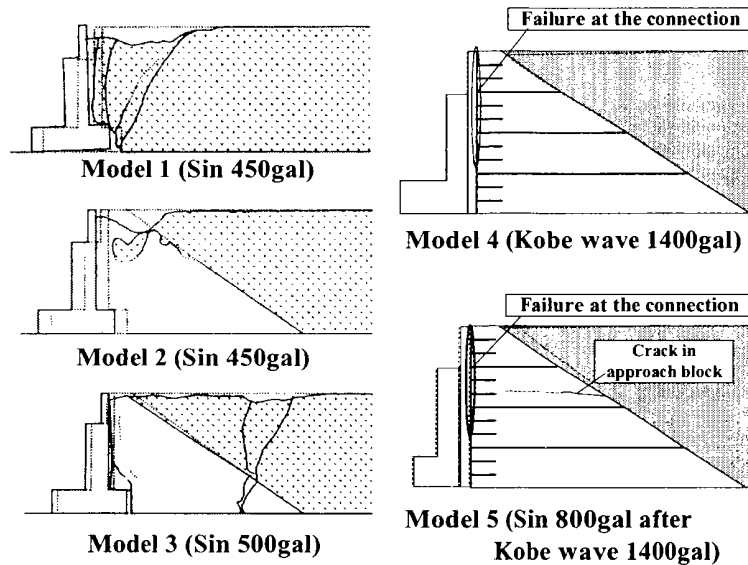


Fig. 23 Deformation of models after tests (Watanabe et al., 2002).

With Models 4 & 5, simulating the new type abutment, the footing base of the facing was either relatively wide (Model 4) or relatively narrow (Model 5). The model reinforcement was a grid made of 0.2 mm-thick and 3 mm-wide phosphor-bronze strips soldered to each other with a center-to-center spacing of 50 cm and 100 cm in the transverse and axial directions. The tensile force in the reinforcement was measured by using electric-resistant strain gauges attached to the central strip at three levels. For all the models, the model ground was made by compacting an air-dried gravel ($U_c = 12.1$; $D_{50} = 10.0$ mm, $D_{max} = 32.0$ mm and $FC = 2.0\%$) to a dry density of 1.9 g/cm³. The dynamic response of the abutment and backfill was measured as shown in Fig. 22. The dynamic earth pressure acting on the back face of the facing and the

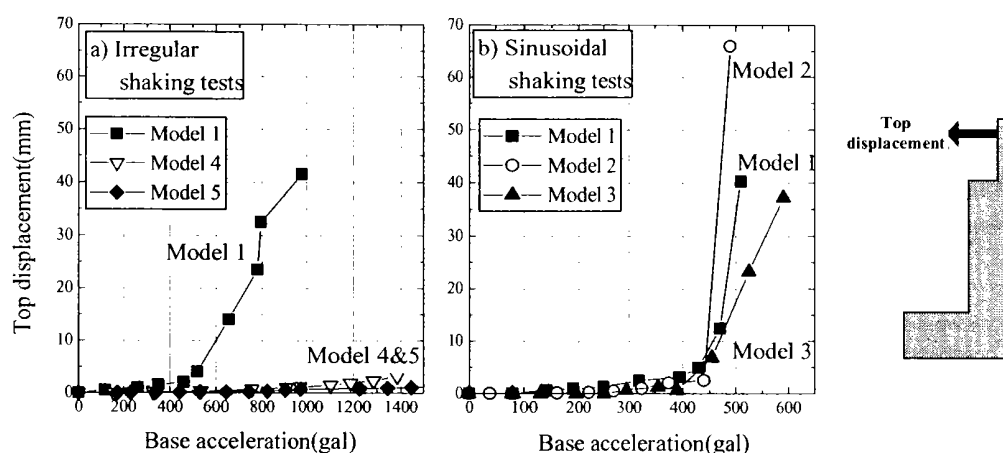


Fig. 24 Residual displacements of model abutments; a) irregular (Kobe wave); and b) sinusoidal table motion (Watanabe et al., 2002).

bottom of the footing of the facing was measured with a number of two-component load cells measuring normal and shear forces.

Models 1, 2 and 3 were subjected to a series of uniform sinusoidal wave, each comprising 50 waves while lasting for 10 seconds at a frequency of 5 Hz. The amplitude of horizontal acceleration at the table, a_{max} , was increased step by step from the initial value, 50 gals, with an increment of 50 gals until the displacements of the abutment became considerably large. Models 1, 4 and 5 were subjected to the time history of horizontal ground acceleration recorded at the Kobe Marine Meteorological Observation Station during the 1995 Hyogo-ken Nambu Earthquake with a predominant frequency adjusted to 5 Hz. The a_{max} value was increased step by step with an increment of 100 gals from 100 gals to 1,400 gals. As Models 4 and 5 did not reach ultimate failure even when a_{max} became 1,400 gals, subsequently sinusoidal waves were applied to the models stepwise increasing a_{max} from 100 gals with an increment of 100 gals until the models exhibited ultimate failure.

Fig. 23 shows the models after the respective test. **Figs. 24a & b** show the residual displacements at the top of the facing plotted against a_{max} at the table for all the models. The deformation of model Model 1 when a_{max} (sinusoidal motion) became 450 gals was very large, showing ultimate failure with a fully developed single major failure plane in the backfill. Model 2 also exhibited brittle failure when a_{max} became 450 gals, showing very large deformation of the approach block of gravel, in particular at the upper part. The deformation, in particular the settlement at the crest of the approach block, of Model 3 when a_{max} became 450 gals was much smaller than that of Models 1 and 2. When a_{max} became 500 gals, the facing started separating from the approach block, exhibiting a high dynamic response, because of no connection strength between them. Several major cracks developed in the approach block of Model 3, resulting into a loss of structural integrity as an abutment. These

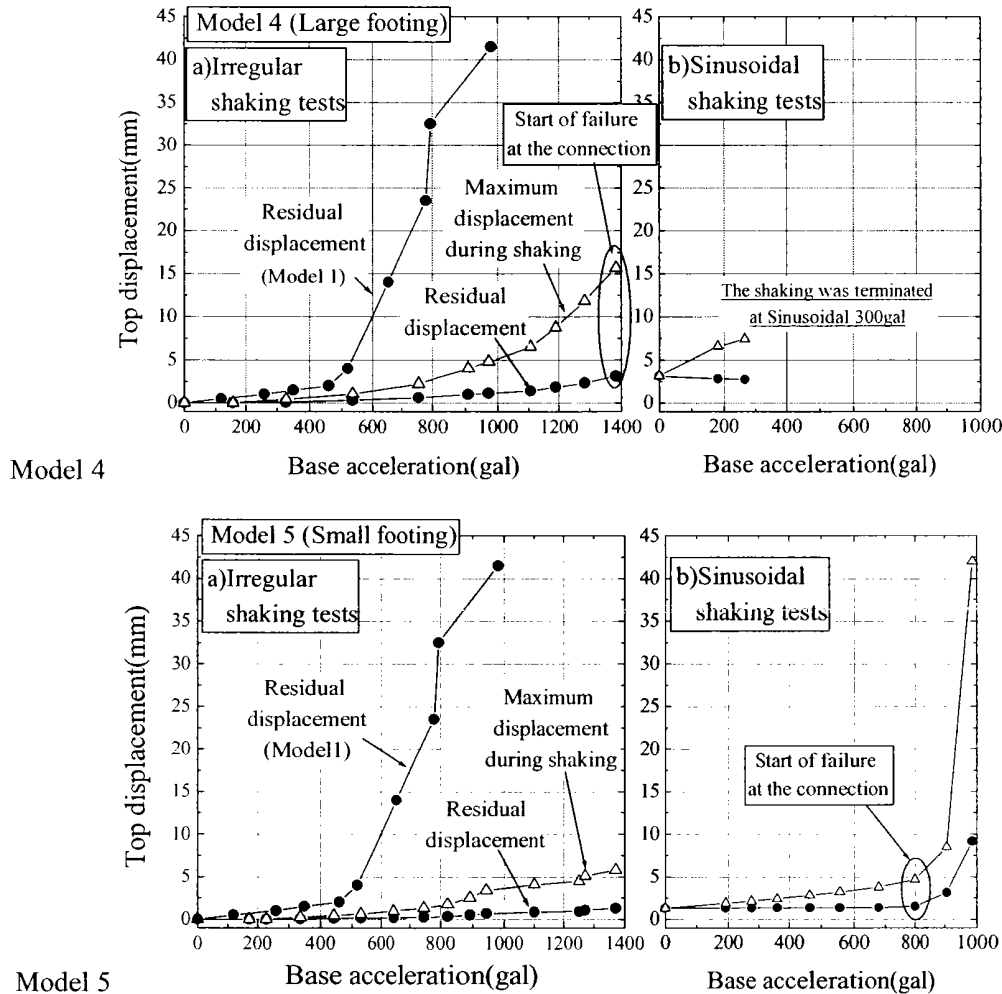


Fig. 25 Displacements of Model 4 and Model 5, compared to that of Model 1 (Watanabe et al., 2002).

results indicate that the seismic stability of these three conventional types of abutment would be insufficient.

Figs. 25a & b show the relationships between the maximum and residual displacement at the top of the facing and the maximum table acceleration, a_{max} , for Models 4 & 5 when subjected to a) irregular waves and b) subsequently sinusoidal input waves. In these figures, the residual displacement at the top of the facing of Model 1 is also presented as reference. It may firstly be seen from Figs. 23, 24 and 25 that Models 4 & 5 were substantially more stable than Models 1, 2 & 3. Secondly, with Model 4, the tensile rupture of the connection started when a_{max} (irregular waves) became 1,400 gals. This phenomenon was also noted by a sudden change in the tensile strain at the respective reinforcement layer. This trend of behaviour was due to larger relative vertical displacements at the interface between the approach block and the facing with Model 4, because of a larger footing width. With Model 5, the connection started failing when a_{max} (sinusoidal wave) became 800 gals, after having been subjected to a series of irregular motion. The fact that Model 5 (with a narrow footing) was more

dynamically stable than Model 4 (with a wider footing) indicates that the backfill is more stable than the facing, particularly under dynamic loading conditions. These results indicate the essential importance of high connection strength for a high seismic stability of this type of abutment.

The test results also showed that, at the dynamically active state when the dynamic component of the lateral displacement at the top of the facing was directing outwards, the resisting components were the reaction force activated near the toe of the footing and the tensile force in the reinforcement. At this state, the contact force near the heel of the footing had decreased, indicating overturning displacements of the facing. Moreover, the dynamic component of the earth pressure at the back of the facing exhibited the minimum value at the dynamically active state, showing that the facing was dynamically less stable than the backfill. On the other hand, the maximum earth pressure was attained at the dynamically passive state. With this type of abutment, therefore, high connection strength between the facing and the reinforcement is essential for a high seismic stability. It is to be noted that these trends of dynamic earth pressure are opposite to those assumed in the conventional seismic design of retaining walls, in which it is considered that the dynamic component of the active earth pressure becomes largest destabilising the retaining walls.

With the prototype bridge abutment, the connection strength between the reinforcement layers of a polymer geogrid and a full-height rigid facing was confirmed by performing full-scale loading tests on as described later in this report.

Table 2 List of specimen for CD TC tested (Watanabe et al., 2003a).

Specimen types	Dry density (g/cm ³)	Confining pressure (kPa)	Water content (%)	Cement/ gravel ratio by weight; $\frac{W_{\text{cement}}}{W_{\text{gravel}}}$ (%)	Cement/ gravel ratio by volume; $\frac{V_{\text{cement}}}{V_{\text{gravel}}}$ (%)	Size of specimen (mm)
Chiba (cement-mixed)	1.8, 2.0, 2.2	0, 20, 50	3.5, 5.0, 8.7, 10.5	2.5	6.78	φ 200* H400
Model Chiba (D < 10 mm) (cement-mixed)	2.0	20	8.7	2.5	6.78	φ 200* H400
Chiba (without cement)	1.8, 2.0, 2.2	0, 20, 80	4.0	-	-	φ 300* H600
Kyushu (cement-mixed)	2.41, 2.56, 2.64	0, 20, 50	4.35	2.5	7.58	φ 150* H300
Kyushu (without cement)	2.41, 2.56, 2.64	0, 20, 50	4.35	-	-	φ 200* H400

Strength of cement-mixed gravel evaluated by large TC tests

The deformation and strength characteristics of compacted cement-mixed sand are controlled by a number of factors (Barbosa-Cruz & Tatsuoka, 1999 & 2000; Kongsukprasert et al., 2001, among others). However, only a limited amount of research has been performed on the use of cement-mixed gravel (CMG) in critical structures requiring a high stability while allowing limited deformation, such as bridge abutments. A series of CD TC tests were

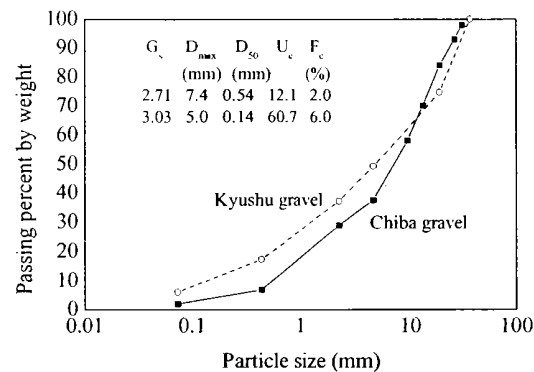


Fig. 26 Two types gravels for CD TC tests (Watanabe et al., 2003a).

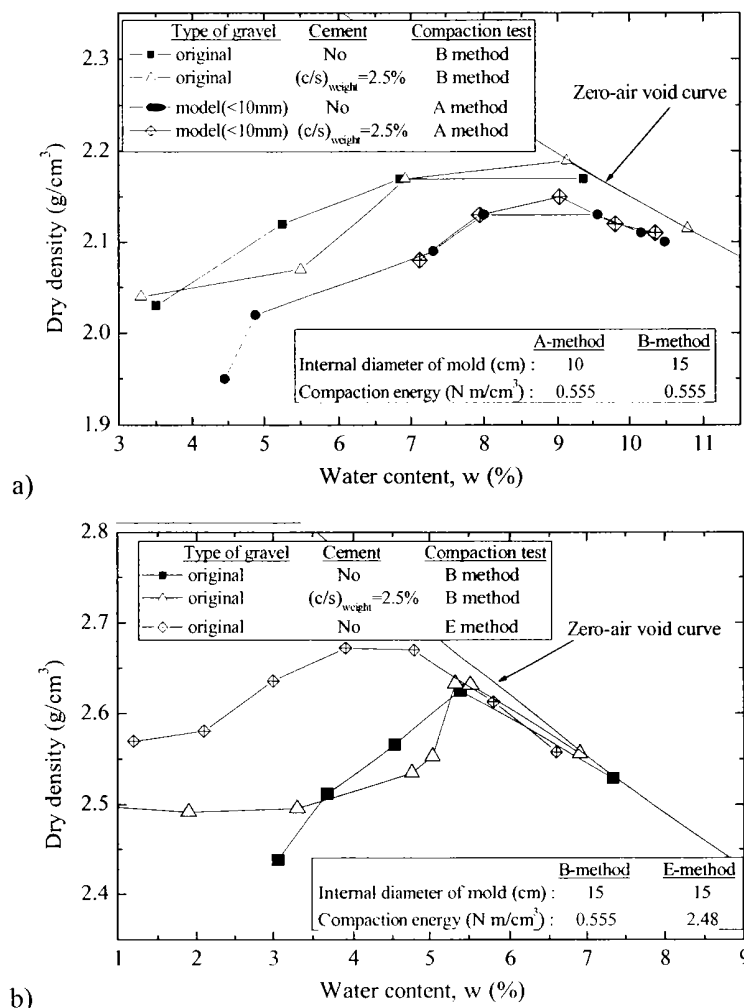


Fig. 27 Compaction curves; a) Chiba gravel; and b) Kyushu gravel.

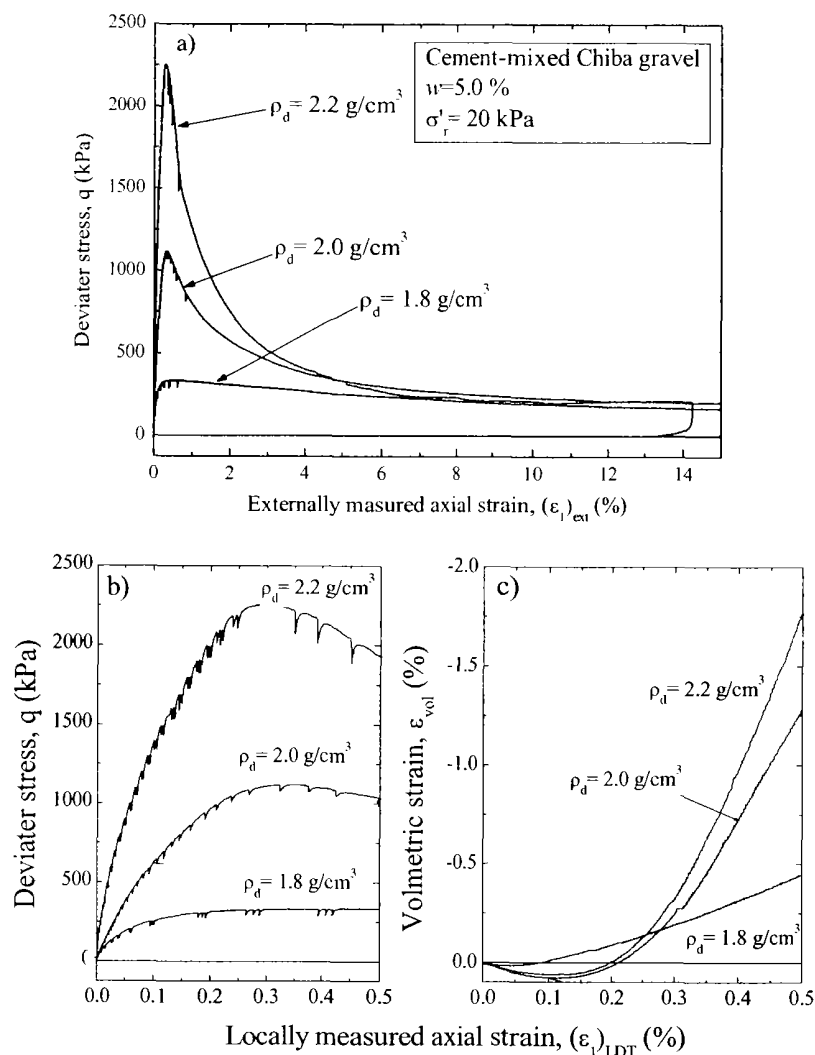


Fig. 28 a) Deviator stress-axial strain (external) relations; b) deviator-axial strain (local) relations; and c) volumetric and axial strain relations, cement-mixed Chiba gravel at different dry densities (Watanabe et al., 2003a).

performed on cement-mixed specimens of two typical types of well-graded quarry gravel (**Fig. 26**); i.e., a crushed sandstone (called Chiba gravel) and a crushed gabbro (called Kyushu gravel), which was used as the backfill of the prototype bridge (Fig. 20).

Fig. 27a shows the compaction curves of cement-mixed and uncemented specimens of the original Chiba gravel and model Chiba gravel (obtained by removing particles larger than 10 mm) obtained by the standard compaction tests using compaction energy of 0.555 N m/cm^3 . **Fig. 27b** is similar results for the original Kyushu gravel. The respective compaction curves of cement-mixed gravel and uncemented one are nearly the same.

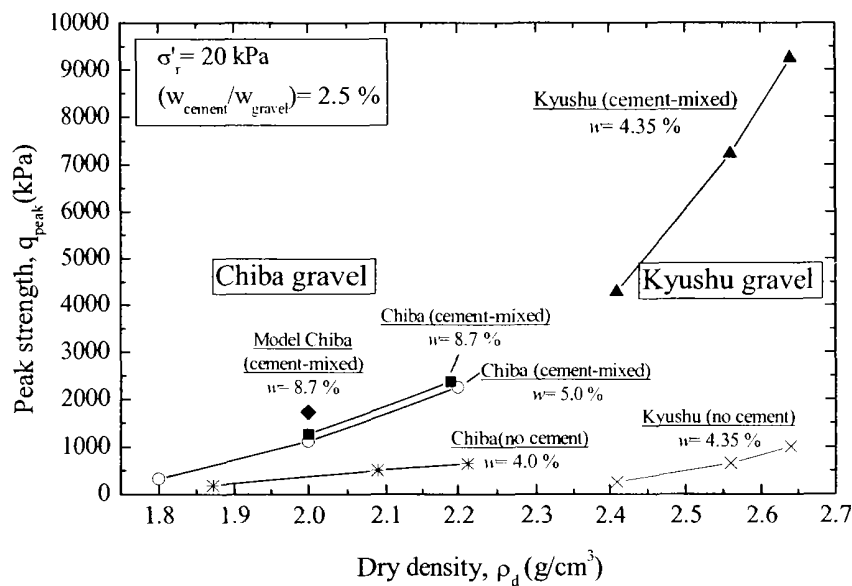


Fig. 29 Relationships between peak strength and compacted dry density of cement-mixed gravels and uncemented gravels (Watanabe et al., 2003a).

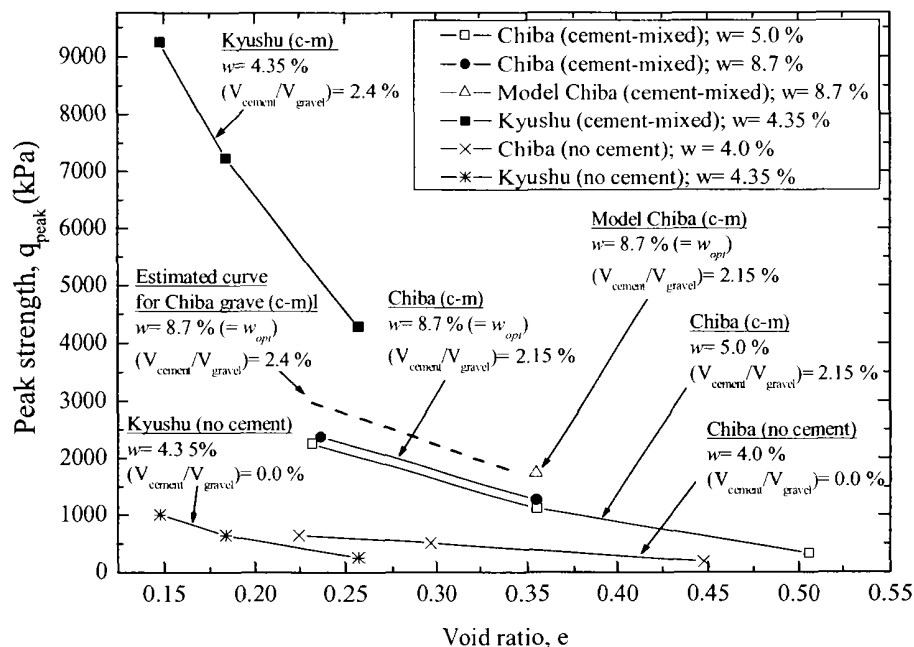


Fig. 30 Relationships between peak strength and compacted void ratio of cement-mixed gravels and uncemented gravels (Watanabe et al., 2003a).

Five types of specimens were prepared for TC tests (Table 2) by manual vertical compaction in about 6 or 12 sub-layers. Different ratios of compacted dry density to the respective maximum dry density for the compaction curves of cement-mixed gravel presented in Figure 27 were prepared (around 80 %, 90 % & 99 % for Chiba gravel and 91 %, 97 % & 100 % for Kyushu gravel). The moulding water content was basically 5 % (dry of optimum) and 8.7 % (optimum) with Chiba gravel, while it was 4.35 % (optimum for compaction by E-method and slightly dry of optimum for compaction by B-method) with Kyushu gravel. To

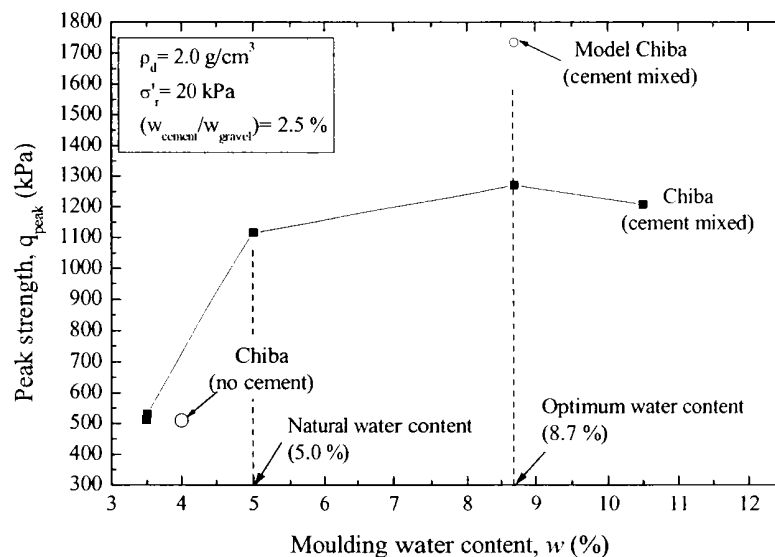


Fig. 31 Relationship between peak strength and moulding water content (Watanabe et al., 2003a)

investigate more in detail the effect of moulding water content on the strength characteristics, the moulding water content was varied between 3.5 % and 12 % with keeping the dry density constant (2.0 g/cm^3) with model Chiba gravel. The moulding water content was 4 % with uncemented Chiba gravel and 4.35 % with uncemented Kyushu gravel. The compacted cement-mixed specimens were cured for 7 days under the atmospheric pressure at constant water content before TC tests. All the CD TC tests were performed using a large triaxial testing apparatus.

Fig. 28 shows typical results from CD TC tests (axial strain rate= 0.01 %/min). Three axial unload-reload cycles with a small axial strain amplitude were applied at various deviator stresses during otherwise monotonic loading (ML) to evaluate the elastic properties. The peak strength increased significantly with compacted dry density of solid materials (gravel & cement), ρ_d . **Fig. 29** summarises the peak strengths plotted against ρ_d . It may firstly be seen that, with both types of gravel, the peak strength of cement-mixed gravel increases with ρ_d at a rate much larger than that uncemented gravel. Therefore, the difference in the peak strength between the cement-mixed and uncemented specimens increases with ρ_d . Secondly, for the same ρ_d , the peak strength of cement-mixed model Chiba gravel, not including particles larger than 9.5 mm and having a smaller coefficient of uniformity, is noticeably larger than that of cement-mixed original Chiba gravel. This is because, for the same ρ_d , the compaction energy is larger with model Chiba gravel.

As seen from Fig. 29, compacted by using nearly the same energy and with the same cement/gravel ratio by weight (W_{cement}/W_{gravel}), the peak strength of cement-mixed Kyushu gravel is significantly larger than that of cement-mixed Chiba gravel. There could be several

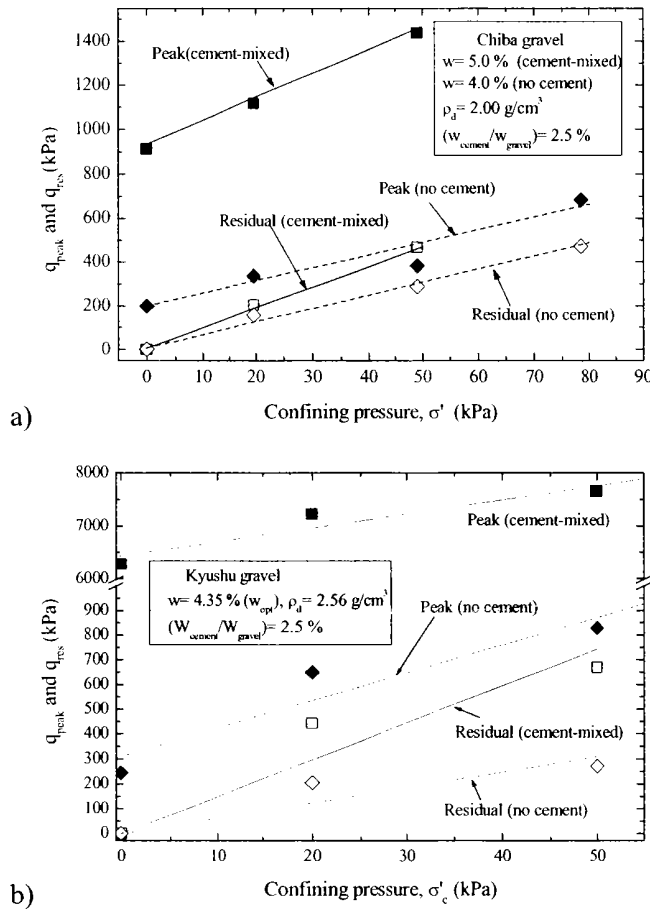


Fig. 32 Relationship between peak and residual strengths and σ'_c ; cemented and uncemented a) Chiba gravel; and b) Kyushu gravel (Watanabe et al., 2003a).

causes for this trend. Firstly, considerably higher ρ_d values, or smaller compacted void ratios (as shown in **Fig. 30**) of Kyushu gravel are due mainly to a specific gravity of gravel particles ($G_s = 3.03$) that is much larger than 2.71 of Chiba gravel. For the same cement/gravel ratio by weight, $W_{cement}/W_{gravel} = 2.5\%$, the cement content ($W_{cement}/\text{specimen volume } V$) is proportional to ρ_d . Therefore, for the relation for the same type of gravel plotted in Figure 3-11, two variables, $\rho_d (= (W_{gravel} + W_{cement})/V)$ and cement content per volume ($= W_{cement}/V$), change simultaneously. On the other hand, G_s itself has no direct link to the packing conditions of particles and therefore to the peak strength of cement-mixed gravel. The peak strength is better linked to the volume parameters: void ratio ($= (V_{gravel} + V_{cement})/V$) and V_{cement}/V . That is, for the same $W_{cement}/W_{gravel} (= 2.5\%)$ and the same void ratio of solid part, the cement/gravel ratio in volume (V_{cement}/V_{gravel}) is different, which is 2.4% (Kyushu gravel) and 2.15% (Chiba gravel). Moreover, the ratios of the moulding waters (4.35% with cement-mixed Kyushu gravel and 8.7% with cement-mixed Chiba gravel) to the respective optimum water content are different. It is shown in the next section that, for the same type of cement-mixed gravel, even at the same compacted dry density, the peak strength becomes largest when compacted at the optimum water content (w_{opt}). In Figure 30, the strength curve

for $V_{cement}/V_{gravel} = 2.4\%$ of cement-mixed Chiba gravel when $w = w_{opt}$ is presented. When compared at the same V_{cement}/V_{gravel} ($= 2.4\%$) and the same ratio $w/w_{opt} = 1.0$, the difference in the peak strength between the two types of cement-mixed gravel at the same void ratio becomes noticeably smaller than when compared at the same W_{cement}/W_{gravel} ($= 2.5\%$) and at the different moulding water content ratios, w/w_{opt} . Yet, the difference is still noticeable. Another factor (or others), such as grading characteristics, is (are) necessary to explain this difference.

Fig. 31 shows the relationships between the peak strength and the moulding water content for cement-mixed Chiba gravel. It may firstly be seen that the peak strength became largest around the optimum water content ($w_{opt} = 8.7\%$). This trend of behaviour is due mostly to the largest compacted dry density (i.e., the smallest compacted void ratio). The other influencing factors include; a) the largest cement content per volume of specimen associated with the maximum compacted dry density (as discussed above); b) the optimum cement paste conditions in terms of the volume of paste and the strength of paste (as discussed below); and c) some specific micro-structure, all attained when compacted at the optimum water content. With respect to the term b, if most of remoulding water is used for the cement paste, the strength of cement paste decreases with an increase in the water content. The decrease in the strength of cement-mixed gravel with an increase in the water content larger than w_{opt} is due essentially to this factor. On the other hand, the peak strength when compacted at $w = 3.5\%$ is very small, close to the peak strength of uncemented gravel. This very low strength was due essentially to an insufficient amount of cement paste to be distributed uniformly at inter-particle contacts. These results indicate that the water content should be strictly controlled, possibly to be at the optimum water content, in compaction in the field.

Figs. 32a & b show the peak and residual strengths plotted against the effective confining pressure, σ'_c , for the cement-mixed gravels with $W_{cement}/W_{gravel} = 2.5\%$ and the corresponding uncemented gravel specimens. The water content was equal to the optimum water content for Kyushu gravel, while it was 5.0% for cemented Chiba gravel and 4.0% for uncemented Chiba gravel. It may firstly be seen that, not only the cohesion intercept, c_{peak} , but also the angle of internal friction, f_{peak} , increase considerably by cement-mixing. This trend of behaviour is in contrast to the case with cement-mixed soft clay, for which f_{peak} decreases by cement-mixing. Secondly, the f_{peak} values of both cement-mixed and uncemented specimens of Kyushu gravel are considerably larger than the respective value of cement-mixed and uncemented specimens of Chiba gravel. The use of these peak strength parameters in the limit-equilibrium type stability analysis may result in an over-estimation of safety factor because of possible large effects of progressive failure. The use of values between the peak and residual strengths would be relevant.

In summary, firstly, the peak strength of compacted cement-mixed gravel could become significantly larger than that of uncemented gravel and the difference increases with an increase in the compacted dry density. This result indicates more significant importance of compaction control with cement-mixed gravel than with uncemented gravel. Secondly, for different types of gravel having different particle specific gravities, the strength of cement-mixed gravel is a function of compacted void ratio, rather than dry density, and cement/gravel ratio by volume, rather than cement/gravel content by weight. The gravel type is another important influencing factor. Thirdly, the strength of cement-mixed gravel becomes the maximum at the optimum water content, due mainly to the maximum compacted dry density (or the minimum compacted void ratio). Finally, not only the cohesion intersect but also the angle of internal friction could significantly increase by cement mixing, unlike cement-mixed clay. Despite that it is much smaller, the friction angle at the residual state could noticeably increase by cement-mixing. With cement-mixed gravel, the residual strength is well correlated to the peak strength.

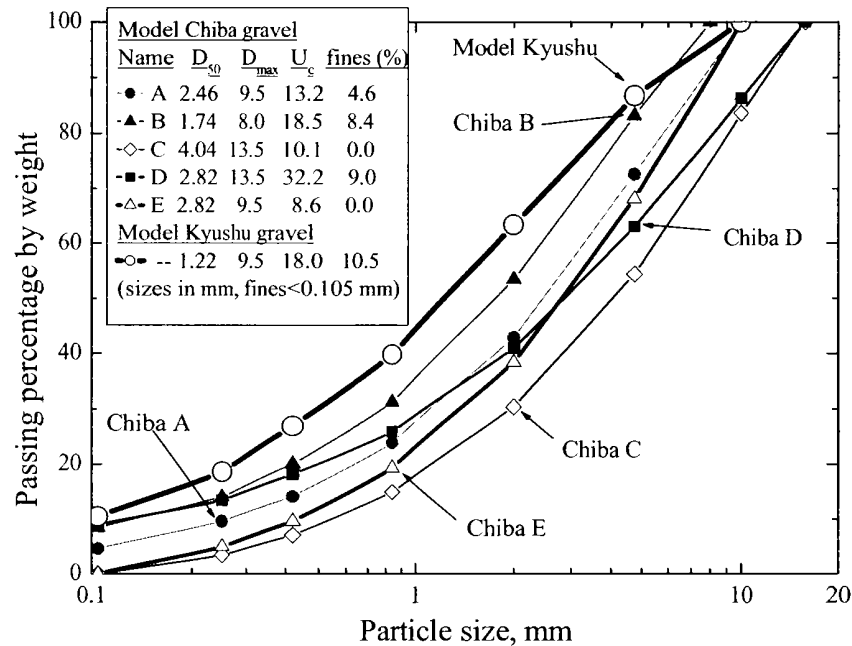


Fig. 33 Gradation curves of the tested materials (Lohani et al., 2003a).

Table.3 Compaction energy levels

Energy	W (kg)	H (cm)	N	B	E (N/cm ³)
E0	2.5	29.5	3	17	0.55
E1	4.5	45.0	5	17	2.55
E2	4.5	45.0	5	34	5.10

Mould (diameter: 7.5 cm & volume: 662.7 cm³), W : rammer weight, H : drop height, N : number of layers, B : blow count per layer, E : applied compaction energy per unit volume.

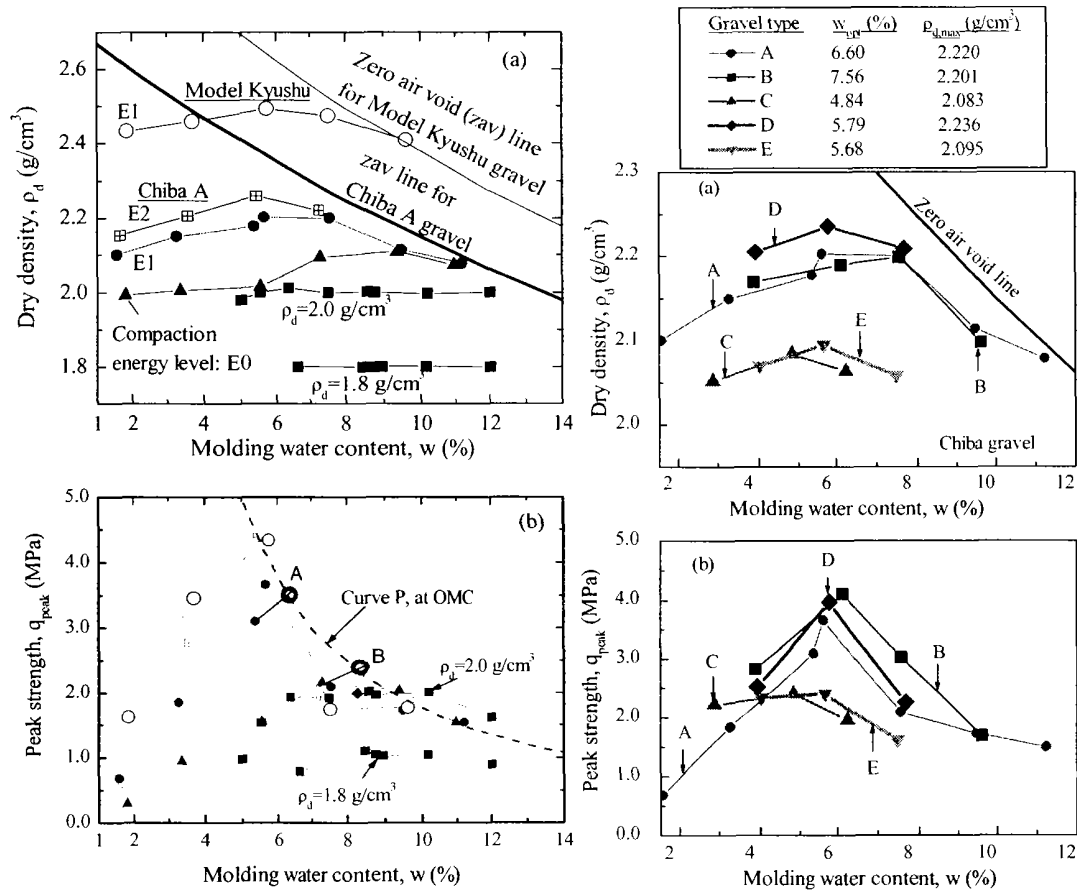


Fig. 34 (left) a) Compaction curves; and b) peak strengths versus moulding water content for different compaction energies, cement-mixed Chiba A and Kyushu gravels.

Fig. 35 (right) a) Compaction curves and b) peak strengths versus moulding water content for Chiba gravels A – E compacted using $E1 = 2.55 \text{ Nm/cm}^3$.

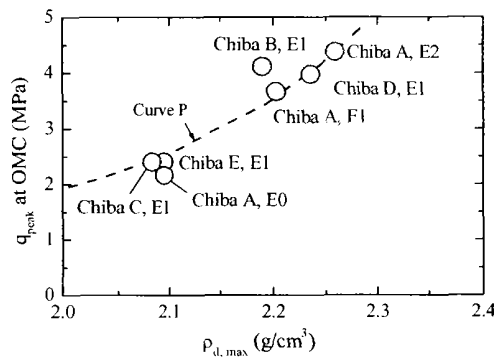


Fig. 36 Relationship between q_{peak} at OMC and maximum dry density for different material types and compaction energy levels (Lohani et al., 2003a).

The effects of moulding water content were studied more in detail using different gravels (Fig. 33). The TC specimens were prepared by two methods:

- Constant dry density:** The specimens were prepared at different water contents by compacting in five sub-layers in a square molds (95 mm x 95 mm x 190 mm) to the

specified target dry densities equal to 1.8 and 2.0 g/cm³. The applied energy for compaction was therefore different among different specimens.

- b. *Constant compaction energy*: The specimens were prepared in a split mold (75 mm diameter x 150 mm height) by applying a prescribed constant level of compaction energy. As listed in **Table 3**, three levels of compaction were employed. The specimens of model Kyushu gravel were prepared using only compaction energy *E1*.

Air-dried gravel was mixed the ordinary Portland cement (2.5 % by weight of gravel) and then after adding a prescribed amount of tap water. The specimens were then cured period of 7 days at constant water content. In the CD TC tests, the confining pressure was 20 kPa and the axial strain rate was 0.03 %/min. Axial and lateral strains were measured locally by using a pair of LDTs and three clip gauges.

Fig. 34a shows the compaction curves of cement-mixed Chiba gravel A obtained by using three compaction energy levels, *E0*, *E1* and *E2* (Table 3-3). With Chiba gravel A, the increase in ρ_d with a decrease in w_{opt} associated with compaction energy is significant, like ordinary unbound geomaterial. The compaction curve of model Kyushu gravel (for energy level *E1*) has the same shape as Chiba gravel A, while the compacted dry density is much higher due to its high specific gravity. In this figure, the data points of the specimens compacted to the two constant dry densities are also shown. **Fig. 35a** shows the compaction curves of Chiba gravels A-E for compaction energy level *E1*. Similar general trends of behavior, as seen in Figs. 27a, may be seen. In addition, the dry densities of materials C & E (without fines) are smallest, while the ρ_d values of Chiba gravel D, having the largest D_{max} and U_c , are highest. This result shows the importance of including some appropriate amount of fines to achieve a high compacted dry density, thereby a high strength.

Fig. 34b shows the relationships between the peak strength, q_{peak} , and moulding water contents, w , corresponding to Fig. 34a (Lohani et al., 2003a). The curve *P* represents the relationship between the maximum value of q_{peak} (at the respective w_{opt} value) and w_{opt} for Chiba gravel A. It may firstly be seen that for the same compaction energy, the peak strength becomes the maximum in the vicinity of w_{opt} . Secondly, even for the same compacted dry density equal to 2.0 g/cm³, the q_{peak} value becomes the maximum at the vicinity of the w_{opt} value for the compaction energy level *E0*, for which the maximum dry density is nearly equal to 2.0 g/cm³. This trend can also be seen with a compaction dry density of 1.8 g/cm³. **Fig. 35b** shows the peak strength for Chiba gravels A-E, corresponding to Fig. 35a. In addition to the general trends seen in Fig. 34b, it may be seen that the peak strengths, q_{peak} , of the materials C & E (without fines) are the lowest, in particular at and around w_{opt} . On the other hand, material D, having the largest values of D_{max} and U_c , exhibited the highest values of q_{peak} .

Fig. 36 shows the relationship between the maximum value of q_{peak} (at w_{opt}) and the

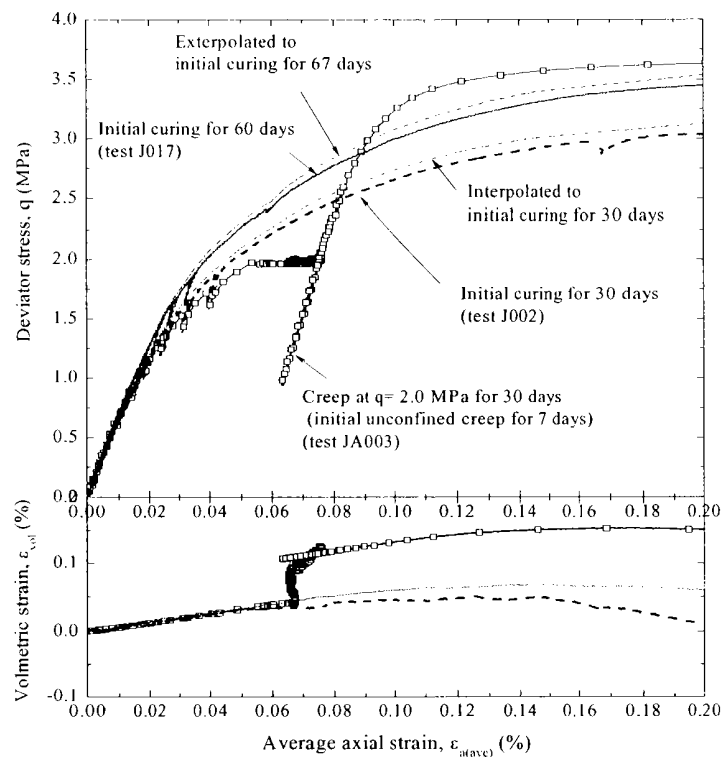


Fig. 37 Effects of drained sustained loading with a shear stress for 30 days on stress-strain behaviour in CD TC (Kongsukprasert & Tatsuoka, 2003).

maximum dry density, $\rho_{d,max}$, from all the data of Chiba gravels and curve P for reference. It may be seen that the q_{peak} value at w_{opt} increases noticeably with $\rho_{d,max}$. This result indicates that it is necessary to select the backfill material that can be compacted to a higher dry density (or a lower void ratio) and to increase the compaction energy as much as possible while seeking for the maximum cost-effectiveness.

Viscous effects and ageing effects

The strength and stiffness of cement-mixed soil increases with curing time like concrete (i.e., the positive ageing effect), while the loading rate effects due to the material viscous properties, including phenomena of creep deformation and stress relaxation, are also important with cement-mixed soil (Barbosa-Cruz and Tatsuoka, 1999; Sugai & Tatsuoka, 2003; Kongsukprasert et al., 2001, Kongsukprasert & Tatsuoka, 2003), like grouted sand (e.g., Delfosse-Ribay et al., 2002). To realistically predict the stress-strain behaviour of cement-mixed soil, therefore, it is essential to properly take into account effects of both ageing and viscous properties. To this end, a series of CD TC tests were performed on unsaturated specimens of Chiba gravel A prepared by *Constant dry density energy* method (described in the preceding section). A part of the test results when compacted around w_{opt} is presented below.

Fig. 37 shows typical test results exhibiting effects of both ageing and viscous properties. The axial strain rate, $\dot{\epsilon}_a$, during drained ML in these tests was 0.03 %/min and sh' was 20 kPa. Significant effects of the initial curing at the atmospheric pressure and constant w_i may be seen on the stress-strain relations from tests J002 and J017 (t_{ini} = 30 days and 60 days). In test JA003 (t_{ini} = 37 days), the specimen was subjected to drained sustained loading at q = 2 MPa for 30 days under otherwise drained ML. It may firstly be seen that the specimen exhibited noticeable creep strain (due to the viscous properties) at the sustained loading stage, while the development of creep strain stopped at an early stage due to the ageing effects. Secondly, the stress-strain behaviour upon the restart of ML at a constant $\dot{\epsilon}_a$ from the sustained loading stage is very stiff and close to the elastic one for a relatively large stress range. Thirdly, upon the restart of ML following the sustained loading in test JA003, the $q - \epsilon_v$ curve over-shot the ML curve for t_{ini} = 67 days before exhibiting clear yielding and then it tended to gradually rejoin the ML curve for t_{ini} = 67 days. The ML $q - \epsilon_v$ relations for t_{ini} = 37 and 67 days were deduced by proportionally changing the measured stress values at t_{ini} = 30 and 60 days. It appears that this overshooting behaviour was due to the development of yield stress during the sustained loading for 30 days, while the development of yield stress was loading history-dependent in that the yield stress for the same total curing time became larger as the deviator stress during curing increased. That is, the stress-strain relation for a given strain rate is not uniquely controlled by the instantaneous total curing time.

The CD TC tests presented above showed that a gravel having a sufficiently high U_c should be used as the backfill while cement-mixed gravel should be compacted by energy as high as possible and at w_{opt} for the specified compaction energy. Moreover, cement-mixed gravel develops a relatively large stress range by drained sustained loading with a shear stress in which the stress-strain behaviour is rather elastic when subjected to loading at a strain rate that is not extremely low. Therefore, the backfill of cement-mixed gravel would exhibit only nearly elastic behaviour when subjected to traffic load. It was concluded that sufficiently strong and stiff backfill of cement-mixed gravel could be constructed as the major structural component of the new type bridge abutment.

The construction of the new type bridge abutment (Fig. 20) started November 2002 and ended two months later. Full-scale field loading tests performed in the end of February 2003. The test results showed that the vertical bearing capacity and the lateral stability of the RC parapet of the abutment are sufficiently high.

Design of the prototype bridge abutment

For the new bullet train line, the design train speed is 260 km/h and the standard design train load (live) load is P-16. That is, 16 tonf or 157 kN is applied to one axle having two

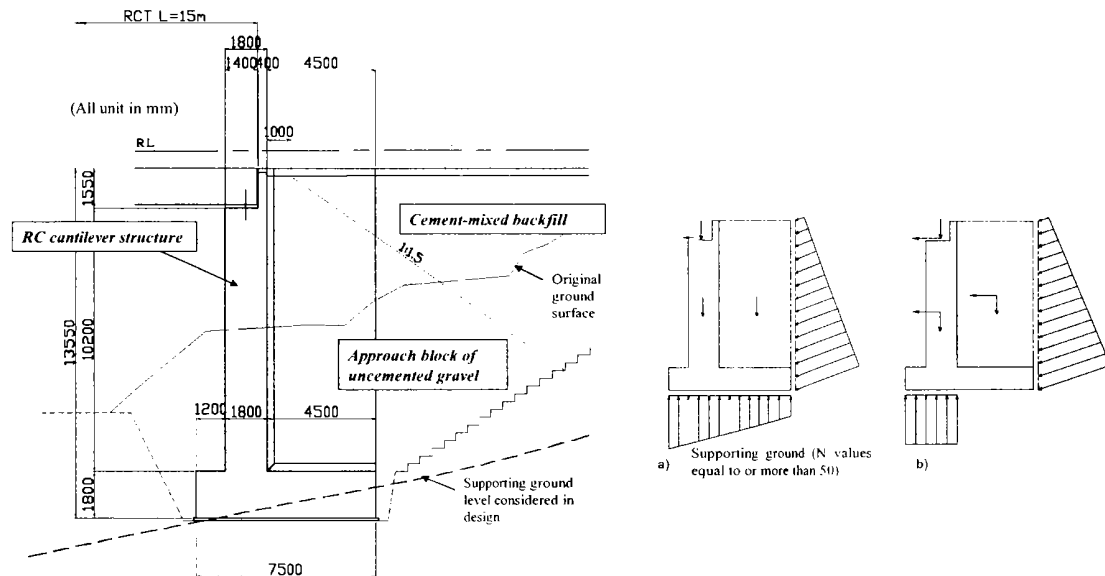


Figure 38 (left) Conventional type bridge abutment designed to be compared with the new type one (Aoki et al., 2002).

Figure 39 (right) a) Static; and b) seismic conditions for conventional type.

wheels. Each pair of axles with a minimum center-to-center distance of 2.2 m supports one bogie, while two bogies with a center-to-center distance of 15 m support one 20 m-long coach. The abutment supports a simple beam T-shaped RC girder having a length of 15 m with a fixed end at the top of the parapet. It was decided that the design compressive strength of cement-mixed backfill is 2 MPa and the design rupture strength of geogrid is 30 kN/m. The soil layers having blow counts (N values) by the standard penetration tests equal to or larger than 50 were considered as the supporting ground of the RC parapet, while the soil layers having N values equal to or larger than 9 were considered as the supporting ground of the approach block (i.e., backfill of cement-mixed gravel).

Fig. 38 shows the conventional type bridge abutment that would have been constructed if the new structural type one were not adopted. The RC facing structure is designed as a cantilever structure resisting against the static earth pressure exerted by the uncemented backfill as well as the static load of the RC structure and bridge girder and the live load (**Fig. 39a**). For seismic design, the dynamic component of earth pressure and the inertia of the RC structure and bridge girder should be additionally taken into account (**Fig. 39b**).

With the new type (**Fig. 20**), on the other hand, the RC parapet supports static and dynamic loads of the parapet and those from the bridge girder and live load (**Figs. 40a & b**). The static and dynamic earth pressure from the backfill is not taken into account considering that the lateral outward movement of the parapet is supported by the cement-mixed backfill under seismic loading conditions. Therefore, the tensile strength of the geogrid layers connecting the

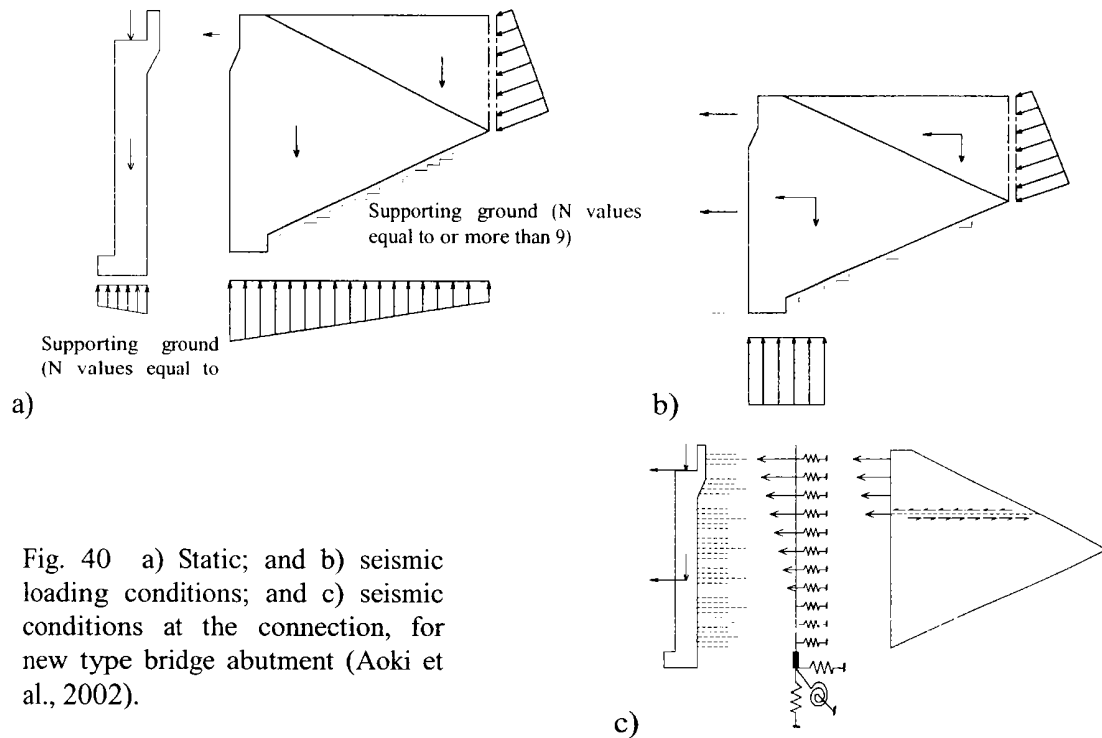


Fig. 40 a) Static; and b) seismic loading conditions; and c) seismic conditions at the connection, for new type bridge abutment (Aoki et al., 2002).

parapet and the backfill is designed to withstand the tensile load activated by the outward lateral movement of the parapet during seismic loading conditions. As shown in **Fig. 40c**, the distribution of tensile force at the connections is obtained by modeling the connections as a series of spring. The approach block of cement-mixed gravel is designed as a gravity-type earth retaining wall resisting against laterals load from the parapet and the earth pressure from the back. Several long layers of geogrid were arranged at a vertical spacing of 0.9 m, prepared for possible development of tension cracks during severe seismic loading conditions. The strength of the long geogrid layers was determined to resist against the seismic tensile force acting at the vertical plane at the back end of the short layers, obtained by assuming that the zone in front of this vertical plane acts as a monolith while ignoring the tensile strength of the backfill.

The construction cost of the new type bridge abutment was estimated to be 87 % of that of the conventional type for the good ground conditions at the site. If the ground conditions are poorer and the conventional type bridge abutment is supported by a pile foundation (as in many cases) while the new type one is not, the difference in the construction cost increases.

Construction

The abutment was constructed by the staged construction method (Fig. 21). The field compaction work was made with a lift of 15 cm to obtain a degree of compaction more than 95 %. A well-graded crushed gravel of gabbro from a quarry (classified as M-40) with $D_{max}=37.5$ mm was used as the backfill. The maximum dry density was equal to 2.68 g/cm^3 at $w_{opt}=$

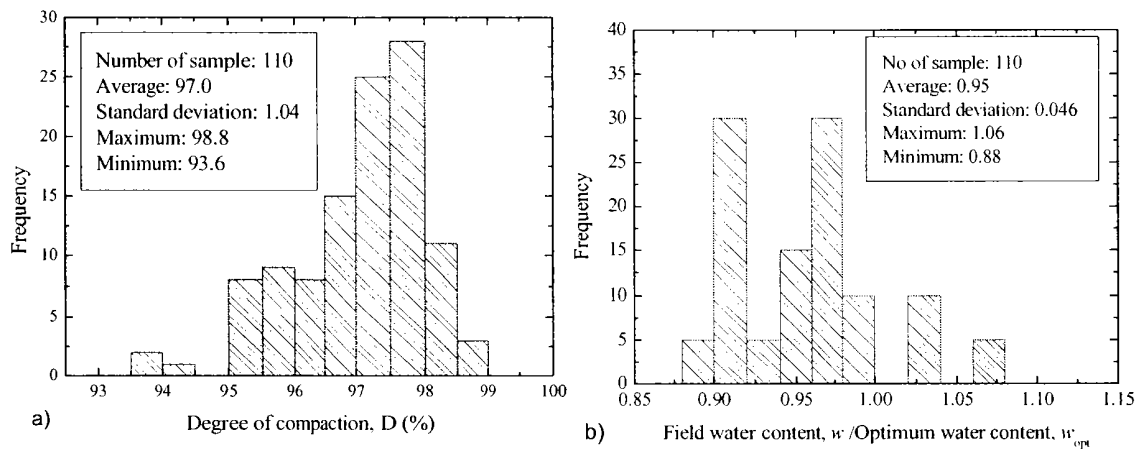


Fig. 41 a) Degree of compaction; and b) water content measured during the construction of backfill (Watanabe et al., 2003b).

4.35 % by the compaction tests using a mould with an inner diameter of 15 cm and an inner height of 12.5 cm for energy level of $2.48 \text{ MN}\cdot\text{m}/\text{m}^3$ (E-b method) (Watanabe et al., 2003b). The specified compaction energy was achieved with a list of 15 cm by ten times passing of a 1 tonf-weight vibratory compaction plant for the first 24 lifts while six times passing of a 4 tonf-weight vibratory plant for the 25th through 81st lifts. A cement-mixed ratio by weight equal to 4 %, which is higher than the value used for the laboratory stress-strain tests, was employed to account for a possible high inhomogeneity of cement mixing in the field. The compacted dry density (by means of RI) and water content measured during construction were satisfactory (Fig. 41a & b).

The backfill was compacted with a help of bags filled with unbound gravel placed at the shoulder of each soil layer (Fig. 21a). As the bags were wrapped around with polymer geogrid layers, the wall face of the completed backfill was covered with geogrid sheets. The RC parapet was then constructed by casting-in-place fresh concrete into a space between an external concrete form supported by steel bars extending from the inside of the backfill and the wall face of the backfill (Fig. 21b). Steel reinforcement had been arranged in the space. Consequently, the parapet was firmly connected to the geogrid layers and gravel bags. By this staged construction method, the following potential problems can be alleviated. Firstly, if the parapet is constructed in advance, excessive earth pressure may be exerted on the parapet when compacting the backfill. To avoid this problem, the zone of backfill adjacent to the parapet may not be well compacted, forming a structurally weak zone. Secondly, if the parapet is constructed in advance with the reinforcement layers connected to the parapet, the connection could be damaged if the settlement of the backfill relative to the parapet during the construction of the backfill becomes excessive. Thirdly, to maintain the alignment of the front face of the complete parapet as constructed, the parapet should be supported with a very stiff propping during the construction of the backfill and/or the footing of the parapet should be

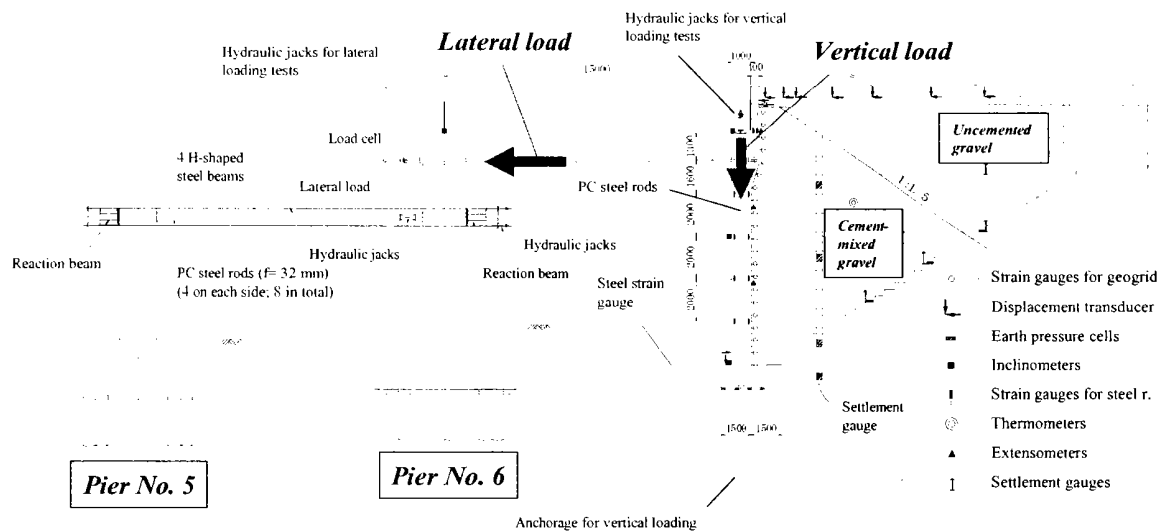
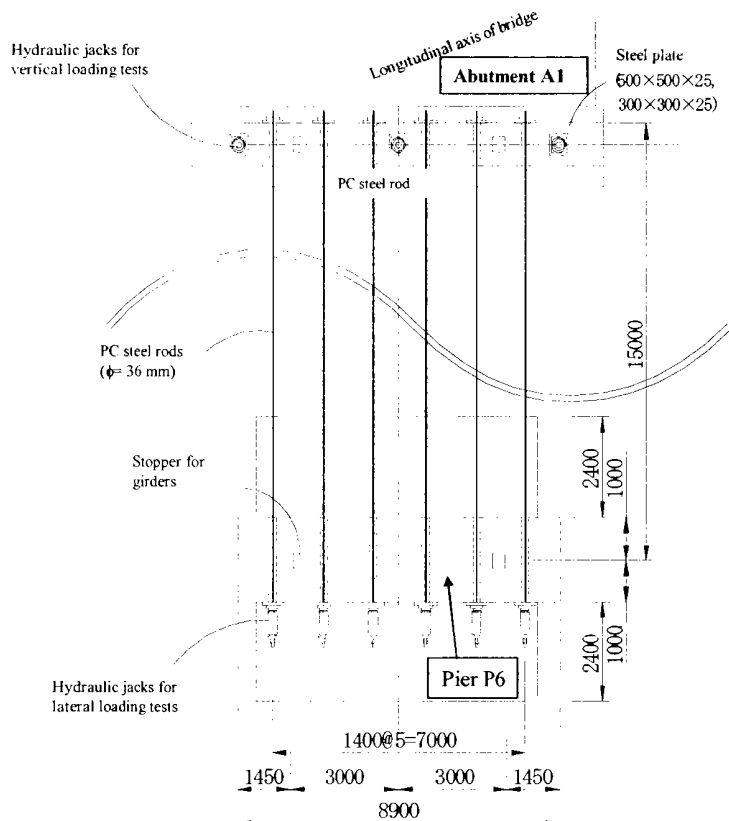


Fig. 42 a) (top) Profile; and b) (right) plan of field full-scale loading test of the bridge abutment with backfill of cement-mixed gravel (Aoki et al., 2003).



made much wider while supported with a pile foundation.

Full-scale loading tests

In the full-scale loading tests on the completed abutment (Fig. 42), first vertical load was applied to the top of the parapet by using three hydraulic jacks set at the shoulder of the parapet with PC steel tendons anchored into the supporting ground. Subsequently, lateral load was applied to the parapet, 1.3 m below the shoulder, by using six hydraulic jacks set at the adjacent pier. Two piers Nos. 5 & 6 were fixed to each other to react against the lateral load (Fig. 42), because it was anticipated that the abutment would be much stronger against lateral

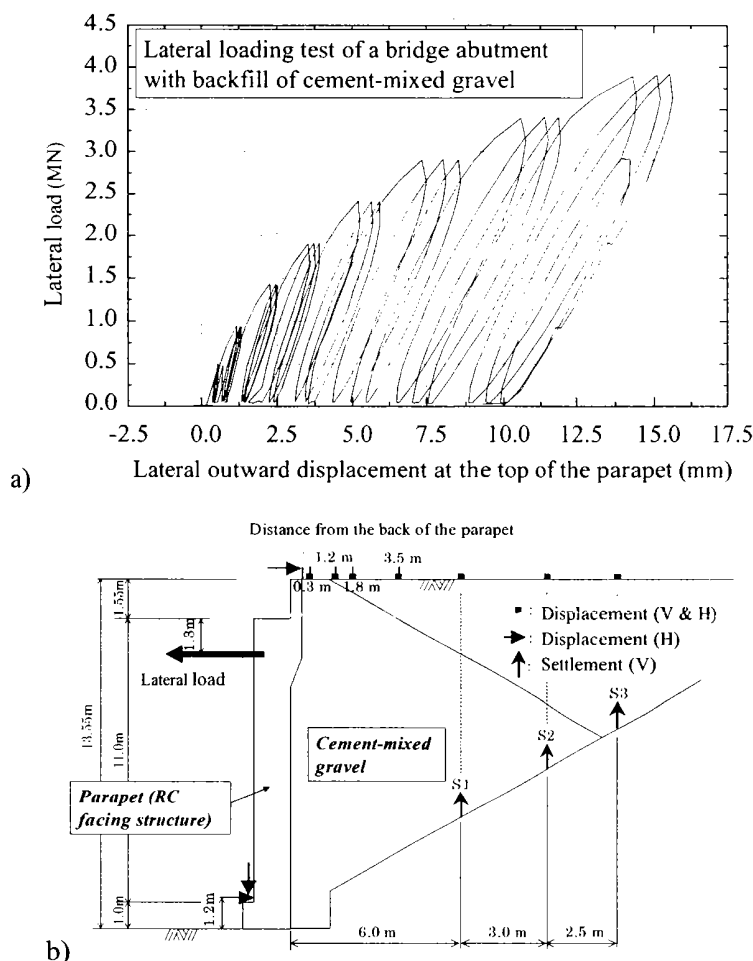


Fig. 43 a) Relationship between lateral load and lateral outward displacement at the top of the parapet; and b) locations of measurement.

loading than a single pier.

The test results showed that, despite relatively small dimensions of the footing for the parapet, the vertical bearing capacity of the footing was sufficiently large as anticipated. **Fig. 43a** shows the relationship between the lateral load and the outward lateral displacement at the top of the parapet. **Fig. 43b** shows the locations of lateral load application and measurement of several quantities. The lateral displacement 1.3 m below the shoulder of the parapet was as small as 15.6 mm at the maximum load (4 MN), which is only slightly lower than the design seismic lateral load (4.171 MN) for a design lateral seismic coefficient of as high as 0.89 corresponding to the design maximum ground horizontal acceleration equal to 871 gals (i.e., so-called level II design seismic load). The residual lateral displacement was 9.3 mm, showing that the behaviour was basically recoverable and the load-displacement state at a horizontal load of 4 MN was far before ultimate failure. The lateral displacements of the two piers was about two times larger than that of the abutment (**Figs. 44a, b & c**), which means that the lateral stiffness of the parapet is about four times as large as that of a single pier. This result indicates that this new type bridge abutment has a lateral stability more than sufficient.

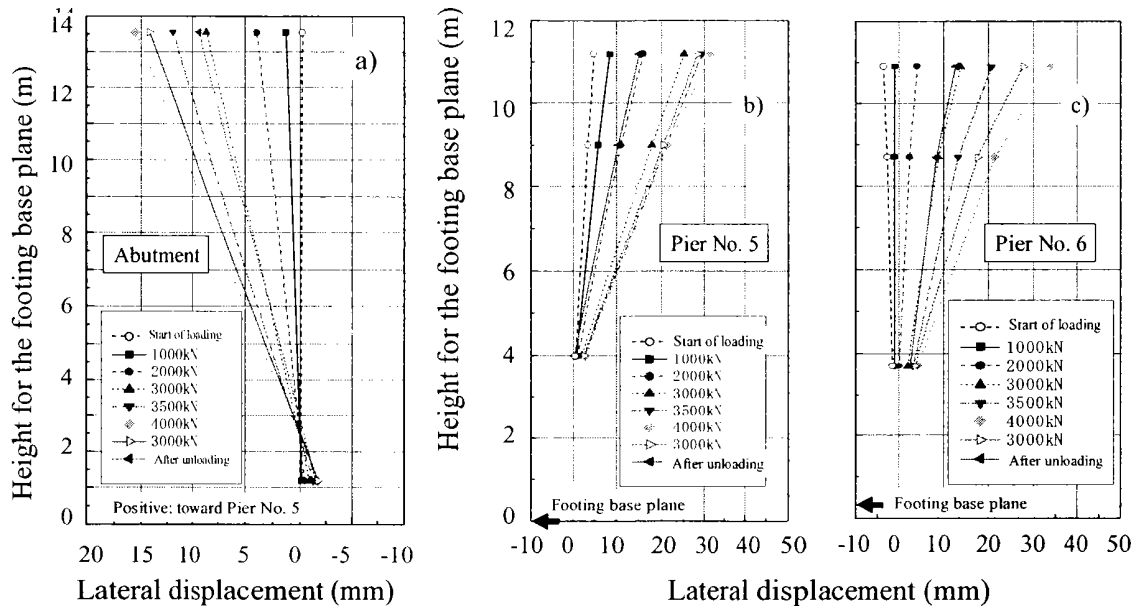


Fig. 44 Lateral displacements of a) abutment; b) pier No. 5; and c) pier No. 6, bridge abutment with backfill of cement-mixed gravel (Aoki et al., 2003).

Figs. 45a & b show the distributions of lateral displacement and settlement on the crest of the backfill (see Fig. 43b for the measuring points). **Fig. 45c** shows the distribution of settlement at the base of the approach block. Firstly, there was no distinct separation between the back of the parapet and the approach block, indicating no significant deformation of the geogrid at the connection. Secondly, the lateral displacement at the crest of the backfill was very uniform until a distance of about 5.5 m, indicating that the approach block behaved like a monolith. A major crack with a width of about 10 mm was found at a distance of 7.25 m from the back face of the parapet at the end of loading. Thirdly, the crest of the backfill exhibited noticeable settlements, which were larger at places closer to the parapet. This trend of behaviour was due mainly to a rigid overturning rotational displacement of the approach block. Correspondingly, the back zone of the approach block exhibited considerable heaving at the base (**Fig. 46**). The changes in the earth pressure along the base of the approach block (**Fig. 47**) as well as the overall displacement of the abutment at the maximum lateral load also indicate that the parapet and the approach block of cement-mixed gravel behaved like a monolith exhibiting a rigid overturning rotational displacement.

Summary

The following conclusions with respect to the new type bridge abutment can be derived from the test results presented above:

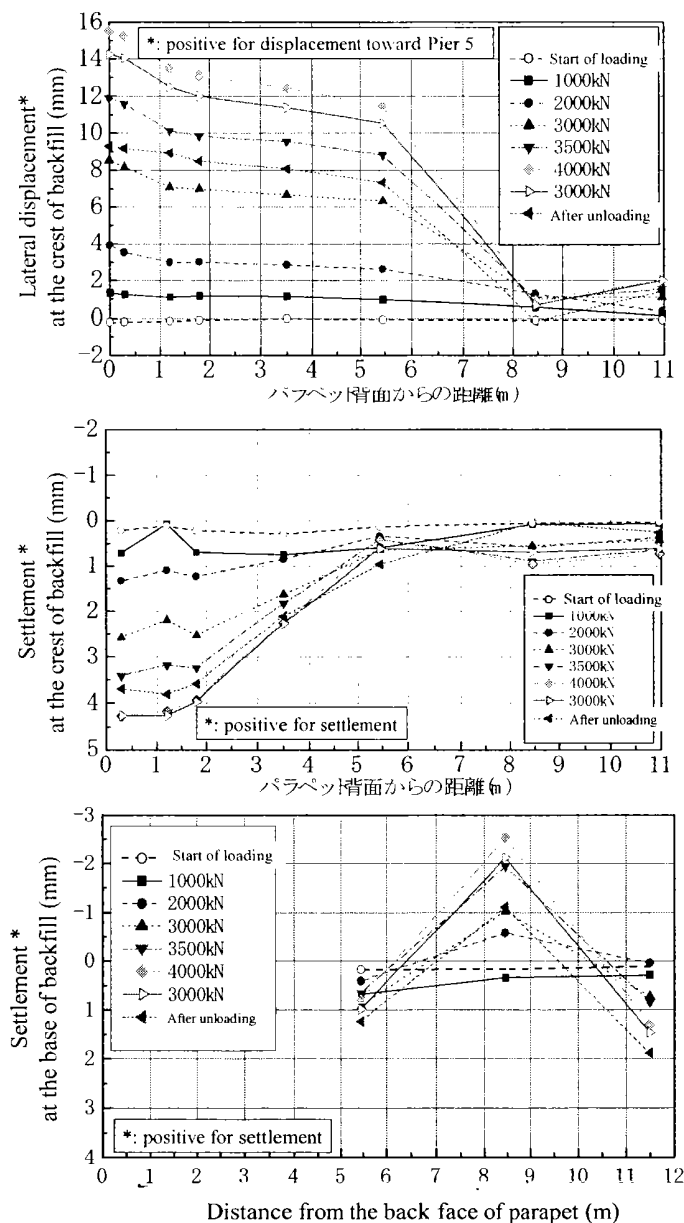


Fig. 45 a) Lateral displacements & b) settlement at the crest; and c) settlement at the base of backfill of cement-mixed gravel crest (Aoki et al., 2003).

- 1) The new type bridge abutment has a lateral stiffness higher by a factor of about four than that of a single pier designed and constructed following the present aseismic design code against severe design seismic load (so called Level II).
- 2) The maximum and residual outward lateral displacements at the top of the parapet when subjected to a very high lateral load simulating severe seismic load were as small as 15.6 mm and 9.3 mm, indicating a very high seismic stability.
- 3) Against very large lateral load, the parapet and backfill behaved like a monolith exhibiting a rigid overturning rotational displacement, showing no noticeable sign of a separation between them.

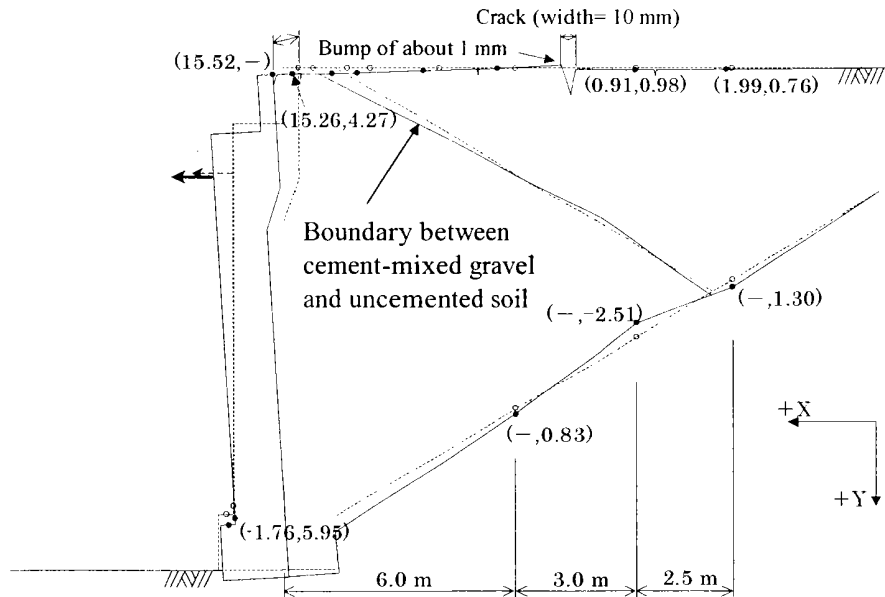


Fig. 46 Overall residual deformation at the maximum lateral load.

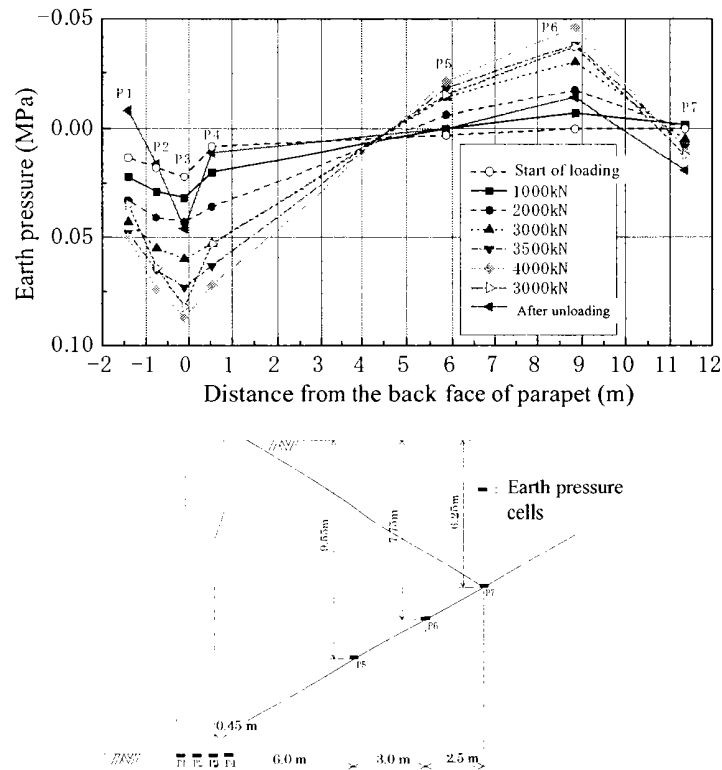


Fig. 47 a) Earth pressure at the base of backfill of cement-mixed gravel crest; and b) locations of earth pressure cells (Aoki et al., 2003).

The advantages of the new type bridge abutment include a high cost-efficiency and a very high seismic stability without showing any serious bump immediately behind the back of front RC structure (i.e., parapet) supporting a bridge girder (or girders). The Japan Railway Construction Public Agency specified “Design and construction standard for bridge abutments having an approach block of cement-mixed backfill” March 2004 to construct new structural

type bridge abutments at other places.

CONCLUSIONS

The following conclusions can be derived with respect to the performance of geosynthetic-reinforced soil structure from the full-scale behaviour and the results from a series of model tests reported in this paper:

- 1) The construction procedure of geosynthetic-reinforced soil (GRS) retaining walls with a full-height rigid facing that is cast-in-place by staged construction procedures has been validated by a number of satisfactory case histories as important permanent structures.
- 2) A new construction method by preloading and prestressing (PL&PS) can make the deformation characteristics of reinforced backfill very stiff and elastic against long-term vertical live load such as traffic load. The long-term performance of a prototype PL&PS GRS bridge pier validated the above.
- 3) Bridge abutments of geosynthetic reinforced backfill become very stable against seismic load by means of prestressing using a tie rods fixed by using a ratchet connection system.

With respect to the new type bridge abutment, it is now possible to construct such permanent structures requiring a high ultimate stability while allowing relatively small displacements by constructing backfill of cement-mixed soil that is reinforced with geosynthetic reinforcement and connecting firmly a RC facing structure (i.e., a parapet) to the backfill with geosynthetic reinforcement. Because of more stable behaviour of the backfill than a slender parapet, unlike the conventional design concept, the backfill does not exert active earth pressure to the RC facing structure. Rather, the backfill supports the parapet, in particular under seismic loading conditions. A proper understanding of the strength and deformation characteristics of cement-mixed soil by systematic and careful laboratory stress-strain tests was essential for the proper design of the structure. Finally, the results from the triaxial compression tests showed that, to achieve a high cost-effectiveness in construction of the backfill of cement-mixed gravel, it is necessary to;

- 1) find the optimum water content for a given backfill type and a specified compaction energy; and
- 2) compact at a water content as much as close to the optimum water content.

REFERENCES

- 1) Aoki, H., Yonezawa, T., Hashimoto, J., Tateyama, M., Yamada, T., Kohara, A. and Tatsuoka, F. (2002). "Comparative design of bridge abutment with backfill of cement-treated soil and conventional type abutment." *Proc. 37th Japan National Conf. on Geotechnical Engineering, Japanese Geotechnical Society*, Osaka, 805-1806 (in

- Japanese).
- 2) Aoki, H., Yonezawa, T., Watanabe, O., Tateyama, M. and Tatsuoka, F. (2003). "Results from field full-scale loading tests on a bridge abutment with backfill of geogrid-reinforced cement-mixed gravel." *Geosynthetics Engineering Journal, Japanese Chapter of International Geosynthetics Society*, 18, 237-2428 (in Japanese).
 - 3) Barbosa-Cruz, E. R. and Tatsuoka, F. (1999). "Effects of stress state during curing on stress-strain behaviour of cement-mixed sand." *Proc. Second Int. Conf. on Pre-Failure Deformation Characteristics of Geomaterials, IS Torino '99* (Jamiolkowski et al., eds.), Balkema, 1, 509-516.
 - 4) Barbosa-Cruz, E. R. and Tatsuoka, F. (2000). "Stress-strain properties from elastic behaviour to peak strength of compacted cement-mixed sand." *Grouting, Soil Improvement, Geosystems including Reinforcement* (Rathamayer eds.), 3-10.
 - 5) Delfosse-Ribay, E., Djeran-Maigre, I., Cabrillac, R. and Gouvenot, D. (2002). "Influence of grouts on unconfined creep behaviour of grouted Fontainebleau sand: experimental study and primary creep modeling." *Ground Improvement*, 6(1), 23-37.
 - 6) Goto, S., Tatsuoka, F., Shibuya, S., Kim, Y.-S., and Sato, T. (1991). "A simple gauge for local small strain measurements in the laboratory." *Soils and Foundations*, 31(1), 169-180.
- Horii, K., Kishida, H., Tateyama, M. and Tatsuoka, F. (1994). "Computerized design method for geosynthetic-reinforced soil retaining walls for railway embankments", *Recent Case Histories of Permanent Geosynthetic-Reinforced Soil Retaining Walls* (Tatsuoka and Leshchinsky eds.), Balkema, pp. 205-218.
- Hirakawa, D., Kongkitkul, W., Tatsuoka, F. and Uchimura, T. (2003): "Time-dependent stress-strain behaviour due to viscous property of geosynthetic reinforcement", *Geosynthetics International*, Vol.10, No.6, pp.176-199.
- Kongkitkul, W., Hirakawa, D., Tatsuoka, F. and Uchimura, T. (2004), "Viscous deformation of geogrid reinforcement under cyclic loading conditions and its model simulation", *Geosynthetics International* (accepted).
- Kongsukprasert, L., Kuwano, R. and Tatsuoka, F. (2001). "Effects of ageing with shear stress on the stress-strain behavior of cement-mixed sand." *Advanced Laboratory Stress-Strain Testing of Geomaterials* (Tatsuoka et al. eds.), Balkema, 251-258.
- Kongsukprasert, L. & Tatsuoka, F. (2003), Viscous effects coupled with ageing effects on the stress-strain behaviour of cement-mixed granular materials and a model simulation, *Proc. 3rd Int. Symp. on Deformation Characteristics of Geomaterials, IS Lyon 03* (Di Benedetto et al. eds.), Balkema, 569-577.
- Ling, H.I., Leshchinsky, D. and Tatsuoka, F. (eds.) (2003), *Reinforced soil engineering, Advances in Research and Practice*, Marcel Dekker, Inc. pp. 512.
- Lohani, T. N. & Tatsuoka, F. (2003). "Effects of molding water content on the strength of cement-mixed gravel." *Proc. of the 12th Asian Regional Conference on Soil Mechanics and Geotechnical Engineering*, Singapore, 1. 497-500.
- Lohani, T. N., Kongsukprasert, L., Watanabe, K., and Tatsuoka, F. (2003a). "Strength and deformation characteristics of cement-mixed gravel for engineering use." *Proc. 3rd Int. Symp. on Deformation Characteristics of Geomaterials, IS Lyon 03* (Di Benedetto et al. eds.), Balkema, 1. 637-643.
- Lohani, T.N., Kongsukprasert, L., Watanabe, K. and Tatsuoka, F. (2003b) "Strength and

- deformation properties of compacted cement-mixed gravel evaluated by triaxial compression tests, *Soils and Foundations* (submitted).
- Nakarai, K., Uchimura, T., Tatsuoka, F., Shinoda, M., Watanabe, K. and Tateyama, M. (2002). "Seismic stability of geosynthetic-reinforced soil bridge abutment", *Proc. of 7th International Conference on Geosynthetics*, Nice, Vol.1 pp.249-252.
- Shinoda, M., Uchimura, T., Maruyama, N. and Tatsuoka, F. (1999). "Effects of preloading and prestressing on the vertical stiffness of GRS structure", *Proc. of 11th Asian Regional Conf. on SMGE*, Seoul, Vol.1, pp.419-422.
- Shinoda, M., Uchimura, T. and Tatsuoka, F. (2003a). "Increasing the stiffness of mechanically reinforced backfill by preloading and prestressing", *Soils and Foundations*, Vol.43, No.1, pp.75-92.
- Shinoda, M., Uchimura, T. and Tatsuoka, F. (2003b). "Model shaking table tests of preloaded and prestressed mechanically reinforced backfill", *Soils and Foundations*, Vol.43, No.2, pp.33-54.
- Sugai, M. and Tatsuoka, F. (2003). "Ageing and loading rate effects on the stress-strain behaviour of a cement-mixed soft clay, *Proc. 3rd Int. Sym. on Deformation Characteristics of Geomaterials*, IS Lyon 03 (Di Benedetto et al. eds.), Balkema, 627-635.
- Tatsuoka F. (1993). "Roles of facing rigidity in soil reinforcing", Keynote Lecture, *Proceedings International Symposium on Earth Reinforcement Practice, IS Kyushu '92*, Balkema, Vol. 2, pp.831-870.
- Tatsuoka, F., Tateyama, M. and Murata, O. (1989). "Earth retaining wall with a short geotextile and a rigid facing", *Proceedings of 12th International Conference on Soil Mechanics and Foundation Engineering*, Rio de Janeiro, pp.1311-1314.
- Tatsuoka, F. and Leshchinsky, D. (eds) (1994). *Recent Case Histories of Permanent Geosynthetic-Reinforced Soil Retaining Walls*, Balkema, pp.349.
- Tatsuoka, F., Tateyama, M. and Koseki, J. (1996a). "Performance of soil retaining walls for railway embankments," *Soils and Foundations, Special Issue for the 1995 Hyogoken-Nanbu Earthquake*, pp.311-324.
- Tatsuoka, F., Koseki, J. and Tateyama, M. (1996b). "Performance of Earth Reinforcement Structures during the Great Hanshin Earthquake", *Special Lecture, Proceedings of International Symposium on Earth Reinforcement, IS Kyushu '96*, pp.537-542.
- Tatsuoka, F., Tateyama, M., Uchimura, T. and Koseki, J. (1997a). Geosynthetic-reinforced soil retaining walls as important permanent structures; Mercer Lecture 1996-1997, *Geosynthetic International*, Vol.4, No.2, pp.81-135.
- Tatsuoka, F., Uchimura, T. and Tateyama, M. (1997b). "Preloaded and prestressed reinforced soil", *Soils and Foundations*, Vol.37, No.3, pp.79-94.
- Tatsuoka, F., Koseki, J., Tateyama, M., Munaf, Y. and Horii, N. (1998). "Seismic Stability Against High Seismic Loads of Geosynthetic-Reinforced Soil Retaining Structures", Keynote Lecture, *Proc. 6th Int. Conf. on Geosynthetics*, Atlanta, Vol.1, pp.103-142.
- Tatsuoka, F., Hirakawa, D., Shinoda, M., Kongkitkul, W. and Uchimura, T. (2004). "An old but new issue; viscous properties of polymer geosynthetic reinforcement and geosynthetic-reinforced soil structures," *Keynote lecture, Proc. GeoAsia04*, Seoul.
- Uchimura, T., Tatsuoka, F., Sato, T., Tateyama, M. and Tamura, Y. (1996). "Performance of preloaded and prestressed geosynthetic-reinforced soil", *Proc. of the International Symposium on Earth Reinforcement* (Ochiai et al., eds.), Balkema, Vol.1, pp.537-542.
- Uchimura, T., Tatsuoka, F., Tateyama, M. and Koga, T. (1998a). "Preloaded-prestressed geogrid-reinforced soil bridge pier", *Proc. 6th Int. Conf. on Geosynthetics*, Atlanta, Vol.2, pp.565-572.
- Uchimura, T., Tatsuoka, F., Tateyama, M. and Koga, T. (1998b). "Behavior of the first prototype

- and full-scale models of PLPS geosynthetic-reinforced soil structure”, *Proc. of Geo-Congress'98, Geosynthetics in Foundation Reinforcement and Erosion Control Systems, ASCE, Boston*, pp.34-48.
- Uchimura,T., Tatsuoka,F, Tateyama,M. and Koga,T. (1998c). “Preloaded-prestressed geogrid-reinforced soil bridge pier”, *Proc. 6th Int. Conf. on Geosynthetics*, Atlanta, Vol.2, pp.565-572.
- Uchimura,T., Shinoda,M., Tatsuoka,F. and Tateyama,M. (2001a). “Performance of PLPS geosynthetic-reinforced soil structure against working and seismic loads”, *Proc. of the 15th International Conference on Soil Mechanics and Geotechnical Engineering*, Istanbul, Vol.2, pp.1633-1636.
- Uchimura,T., Tatsuoka,F., Sato,T., Tateyama,T. and Tamura,Y. (2001b). “Performance of preloaded and prestressed geosynthetic-reinforced soil”, *Proc. Int. Symp. on Earth Reinforcement*, Fukuoka, (Ochiai et al., eds), Balkema, Vol. 1, pp.537-542.
- Uchimura,T., Tatsuoka,F., Shinoda,M., Sugimura,Y. and Kikuchi,T. (2002). “Roles of tie rods for seismic stability of preloaded and prestressed reinforced soil structures”, *Proc. of 7th International Conference on Geosynthetics*, Nice, Vol.1, pp.115-118.
- Uchimura,T., Tatsuoka,F. and Nakarai,K. (2003a). “Seismic stability of preloaded and prestressed reinforced soil abutments”, *Proc. 12th Asian Regional Conference on Soil Mechanics and Foundation Engineering*, Singapore August.
- Uchimura,T., Tateyama,M., Koga,T., and Tatsuoka,F. (2003b). “Performance of a preloaded-prestressed geogrid-reinforced soil pier for a railway bridge”, *Soils and Foundations*, Vol.43, No.6, pp.33-50.
- Uchimura,T., Tatsuoka,F. and Tanaka,I. (2003c). “Long-term residual deformation of prototype geotextile-reinforced gravel structures”, *Proc. 3rd Int. Symp. on Deformation Characteristics of Geomaterials*, IS Lyon 03 (Di Benedetto et al. eds.), Balkema, Sept. pp.1353-1361.
- Watanabe, K., Tateyama, M., Yonezawa, T., Aoki, H., Tatsuoka, F. and Koseki, J. (2002). “Shaking table tests on a new type bridge abutment with geogrid- reinforced cement treated backfill.” *Proc. of 7th Int. Conf. on Geosynthetics*, IGS, Nice, 1, 119-122.
- Watanabe, K., Tateyama, M., Jiang, G, Tatsuoka, F., and Lohani, T. N., (2003). “Strength characteristics of cement-mixed gravel evaluated by large triaxial compression tests.” *Proc. 3rd Int. Symp. on Deformation Characteristics of Geomaterials*, IS Lyon 03 (Di Benedetto et al. eds.), Balkema, 683-693.
- Watanabe, O., Nasu, Y., Aoki, H., Yonezawa, T., Tateyama, M. and Tatsuoka, F. (2003b). “Construction control of approach block of cement-mixed gravel backfill.” *Proc. 58th Annual Conf. of JSCE, Tokushima, III*, 1091-1092 (in Japanese).

ACKNOWLEDGEMENTS

The author sincerely acknowledges all the help and cooperation provided by his previous and present colleagues to write this report.

DEVELOPING FINAL SET TABLES FOR LONG DRIVEN PILES USING WAVE EQUATION ANALYSIS

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K. H. Tang² and H. W. Pang²

Abstract: Satisfactory driving criteria should ensure that an installed pile has an adequate capacity and the pile can be driven safely at a reasonable production rate with an appropriate hammer system. A traditional way to develop driving criteria is the use of the Hiley formula or its variations. Yet the Hiley formula only addresses pile capacity issues and is known to be inadequate for very long piles. The wave equation approach provides a means with which most aspects of pile driveability can be addressed. This paper outlines a methodology to develop final set tables for long driven piles using wave equation analysis. The proposed procedure has been applied to a site. Two preliminary steel-H piles approximately 60 m long were installed based on the final set table developed using the wave equation. The test piles were subsequently load tested following the procedure outlined by PNAP66. The performance of the test piles during the static load test was satisfactory. Both the pile settlement at twice the design load and the residual settlement are well below allowable values. Hence, the proposed methodology can provide a cost-effective yet safe solution to long piles.

COMMON PRACTICE IN PILE DRIVING

The current practices of driven piles in Hong Kong have been discussed by GEO (1996), Ng et al. (2000), Pratt et al. (2000), Li et al. (2003) and others. Final set tables for pile driving control are usually derived based on dynamic formulas such as the Hiley formula and the HKCA formula (HKCA 1995). These formulas estimate the ultimate bearing capacity of a pile considering the efficiency of the hammer, the temporary elastic compressions of the ground, the pile and the helmet, and the final set.

Dynamic formulas suffered from some inherent problems. Most driving formulas including the Hiley formula are derived based on some of the following assumptions (Poulos and Davies 1980; Tomlinson 1995; GEO 1996; Zhang et al. 2003):

- (1) Conservation of energy and momentum apply;
- (2) The pile is rigid;
- (3) The soil behavior is elastic-perfectly-plastic;
- (4) The static and dynamic pile resistances are the same; and
- (5) The set-up effect is ignored.

These assumptions lead to poor representation of the driving system, system energy losses, and soil behaviour. Abe and Thendean (1996) measured energy transfer ratios using pile driving analyzers (PDAs). The energy transfer ratio can vary in a large range and is a function of, among other factors, the pile material and the hammer type (Fig. 1). The energy transfer ratio may also change in the process of driving a pile when cushion properties change and when the pile penetrates through different soil layers. A dynamic formula does not provide the mechanics to describe these changes. As a result, a dynamic formula can overestimate energy transfer when piles are short or when steam hammers and concrete piles are involved (see Fig. 1). On the other hand, a dynamic formula can underestimate energy

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transfer involving modern hydraulic hammers and long steel piles. As such, it is often difficult to use the Hiley formula to produce workable final set tables for long piles with modern hydraulic hammers. The hydraulic hammer factor K_h values established by HKCA in 1995 may also not be suitable for modern hydraulic hammers. The only way to achieve the final set requirement, according to the Hiley formula, is to increase the hammer energy. For piles longer than 70 m, drop hammers as heavy as 18.5 tons, lifted to a drop height greater than 3.3 m had to be adopted. This might result in pile damage (Gammon 1982; GEO 1996). Thus, piling contractors are often forced to use modern efficient hydraulic hammers for pitching of piles and less efficient drop hammers for final setting of piles.

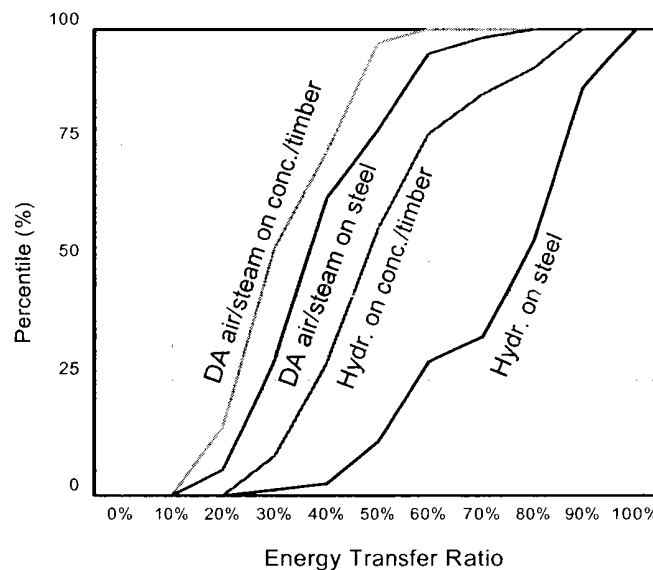


Fig. 1. Energy transfer ratio with various piles and hammers (After Abe and Thendean 1996)

INTRODUCTION TO WAVE EQUATION ANALYSIS

Wave equation analyses utilize the one-dimensional wave equation to simulate the pile driving process and to estimate pile stresses and the pile capacity. In the wave equation, the driving equipment, the pile and the soil are modeled as a discrete set of masses, springs and viscous dashpots as shown in Fig. 2. The pile spring stiffness is calculated from the cross sectional area and modulus of elasticity of the corresponding pile section. Hammer cushions are represented by additional springs whose stiffnesses are calculated from the area, modulus of elasticity, and thickness of the cushion material. The soil resistances along the embedded portion of the pile and at the pile toe are represented by a static component and a dynamic component. The static soil resistance is modeled by an elasto-plastic spring and the dynamic soil resistance is simulated by a linear viscous dashpot.

Smith (1960) developed the first computer solution to the wave equation. The theories were later further advanced and implemented in practice by several researchers such as Goble and Rausche (1976) and Rausche et al. (1985). Final set tables for various types and lengths of piles can be developed using efficient computer programs such as GRLWEAP (Goble Rausche Likins and Associates 2002).

Many shortcomings associated with dynamic formulas can be avoided by realistically simulating hammer impact and the pile penetration process. A wave equation analysis program has a built-in hammer database and the wave equation can be calibrated to suit various types of hammer currently used on site. The wave equation can also provide indicators such as the maximum compressive stress, the maximum tensile stress and

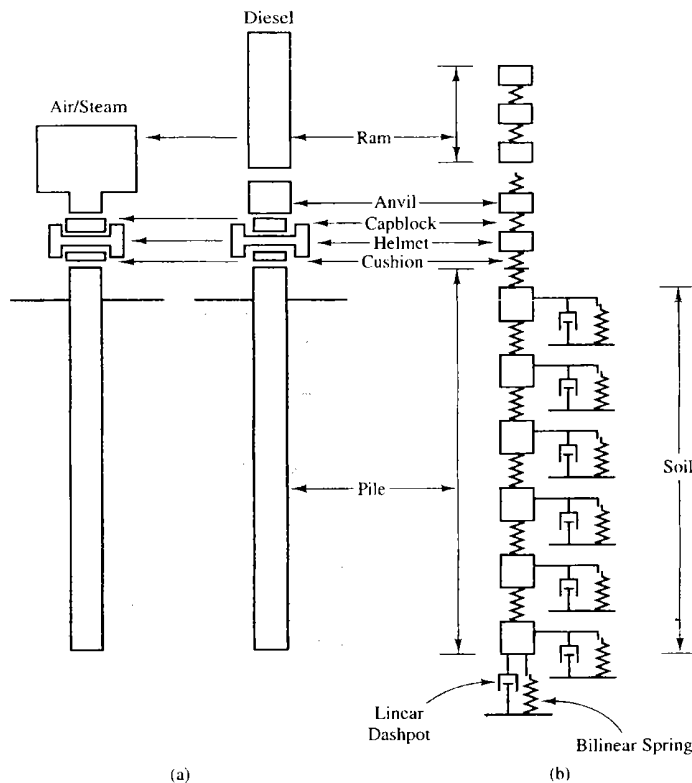


Fig. 2. Conceptual model used in wave equation analysis: (a) actual system; (b) numerical model (After Goble and Rausche 1976)

the hammer efficiency that are essential for evaluating the performances of the pile and the hammer. Hence, various types of piles can be driven more safely to specifications.

A comparison of the capabilities of the wave equation and a pile driving formula is presented in Table 1. A single wave equation analysis can solve many problems anticipated during a pile driving process. In contrast, a dynamic pile driving formula can perhaps answer questions (1) and (2) regarding the pile capacity, but may not be able to address the remaining important problems.

Table 1. Comparison of capabilities of wave equation and dynamic formulas

Engineering problems	Wave equation	Dynamic formulas
(1) Would the pile have sufficient bearing capacity at a specified set?	Yes	Yes
(2) Can the pile be driven at a reasonable blow count to required capacity? (e.g. larger than 2.5 mm/blow)	Yes	Yes
(3) Would heavier pile section or a higher strength pile material allow for easier and/or safer installation?	Yes	No
(4) What stresses will develop along the length of non-uniform piles or piles with splices or joints?	Yes	No
(5) Will intermediate soil layer cause refusal or high stresses?	Yes	No
(6) Will it be possible to drive the pile after driving interruptions and can the pile be moved during a restrike test?	Yes	No
(7) Can a cheaper, smaller or a larger, faster hammer be used?	Yes	No
(8) How do diesel, hydraulic, air-steamed hammers compare?	Yes	No
(9) How much time will be required to drive the pile?	Yes	No

DEVELOPMENT OF FINAL SET TABLES FOR LONG PILES

Grade 55C 305x305x223 kg/m steel-H piles founded on dense CDG/HDG are considered in this study. The required pile capacity corresponding to a safety factor of 2.0 is 7096 kN. A methodology of developing final set tables for pile driving control using the wave equation is proposed and shown in Fig. 3. The wave equation program GRLWEAP was used to develop final set tables for the piles. The soil parameters for wave equation analyses were derived from a correlation study following a procedure proposed by Thendean et al. (1996) and USDOT (1996). In that study, the pile force, the velocity and the transferred energy measured from PDAs were compared against computed values at the first pile segment using GRLWEAP. Adjustments and calibrations to the wave equation input parameters were then made to fit the measured values. Table 2 lists values recommended for long piles founded in dense CDG. These parameters do not differ greatly from those commonly used overseas (Goble Rausche Likins and Associates 2002).

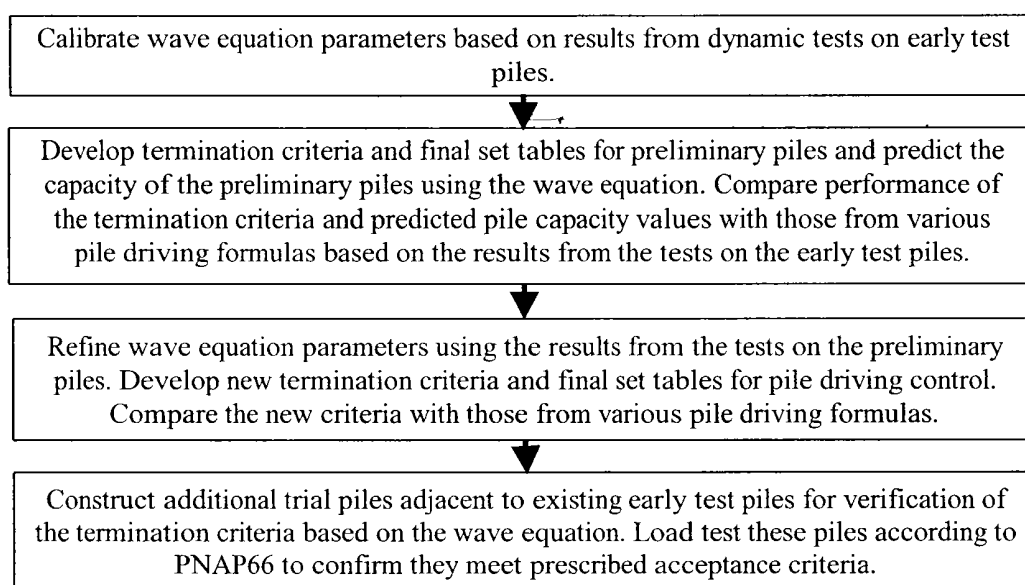


Fig. 3. Proposed methodology of using the wave equation in developing final set tables

Table 2. Soil parameters for wave equation analysis

	Average parameters for piles in dense CDG	Recommended parameters (Goble Rausche Likins and Associates 2002)
Shaft quake (mm)	2.5	2.5
Toe quake (mm)	2.5	2.5
Shaft damping factor (s/m)	0.27	0.16-0.65
Toe damping factor (s/m)	0.24	0.5

Using GRLWEAP and the parameters in Table 2, final set tables for piles can be readily developed and presented in a format similar to those based on Hiley's formula. Table 3 shows a sample final set table for 305x305x223 steel-H piles driven by DKH1523 hydraulic hammers. The predicted temporary pile compression and the transferred energy are for reference purposes. The predicted temporary pile compression increases with the pile length but the energy transfer ratio does not change considerably with the pile length. When the pile is very long, the allowable set value does not decrease considerably with a further increase in pile penetration.

The engineer may want to compare final set tables developed with the wave equation approach with those developed using various pile driving formulas. Fig. 4 compares the allowable final set values calculated using the HKCA formula and the wave equation. In the HKCA formula, a hydraulic hammer factor of 0.70 is used following HKCA (1995). Fig. 4 also shows the final setting results of approximately 700 piles at a site using DKH1523 hydraulic hammers. The HKCA formula was the primary termination criterion used for accepting these piles. As can be seen from the two figures, the allowable set values are unreasonably low when the elastic compression values are greater than 60 mm. In addition to overstress issues, some piles could not be accepted at all as they were “out of the range” of the set table. With the wave equation approach, piles of different lengths can be final set with reasonable allowable set values. When the piles are long or the ground conditions are strong, the wave equation approach allows larger final set values than the HKCA formula. In addition, the allowable sets from the wave equation are not sensitive to a slight change in the hammer energy and the pile length. For example, the differences in the allowable set values for long piles are less than 7 mm when the transferred energy ratio changes from 78% to 82%. Note that it is the type of long piles and strong ground conditions that really caused pile integrity and construction delay problems to the contractors. It can be envisaged that the use of the wave equation approach can resolve many of the common problems in construction sites.

Table 3. Sample final set table for 305 x 305 x 223 steel-H piles

Wave equation analysis program		GRLWEAP 2003		
Hammer model		DKH 1523		
Weight of ram		238.38	kN (24.3 tons)	
Drop distance of ram		1.5	m	
Pile type		305 x 305 x 223	kg/m	
Assumed shaft resistance percentage		70	%	
Adopted coefficient of restitution of hammer cushion		0.9		
Weight of helmet		27.4	kN	
Design hammer energy transfer ratio*		0.78		
Required pile capacity		7096	kN	

Pile length (m)	Allowable final set (mm / 10 blows)	Predicted elastic compression (mm)	Predicted transf. energy (kJ)	Energy transf. ratio (%)
35	95	45	278	78
40	77	48	278	78
45	57	50	278	78
50	37	52	278	78
55	26	54	278	78
60	25	56	280	78
65	23	58	280	78
70	22	59	281	78

* To be confirmed by PDA measurements

VERIFICATION OF WAVE EQUATION IN HONG KONG

To verify the performance of piles final set using the wave equation approach, two preliminary piles, PP-1A2-2 and PP-1B3-1, were installed and tested recently at a site. In addition to verification using the test results from the two preliminary piles final set by the wave equation approach, wave equation predictions for more than 20 preliminary piles final set by other methods at two construction sites were also performed and the predicted results were compared against static load test results of these piles.

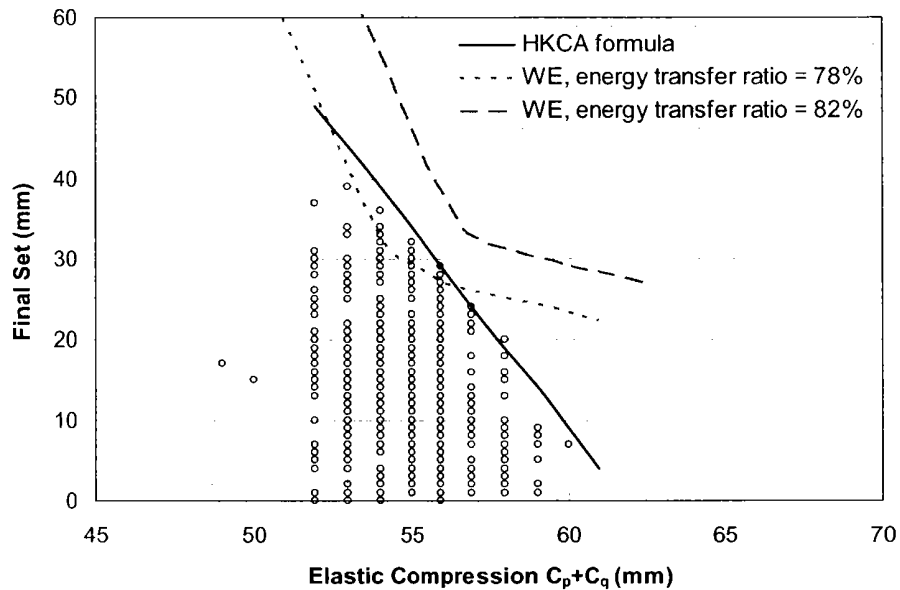


Fig. 4. Comparison of final setting of 706 piles at a site with DKH1523 hydraulic hammers

Fig. 5 shows the soil profiles drawn from boreholes near the two preliminary piles for wave equation study. Three major soil layers were encountered. The surface fill layer was 9.5 m thick. It is underlain by a thick alluvium. The completely decomposed granite is found at depths between 24.5 m and 29.4 m. The blow count, N , values from the standard penetration test (SPT) are also shown in Fig. 5. The SPT- N values are greater than 200 at a depth of approximately 34 m near PP-1A2-2 and 47 m near PP-1B3-1.

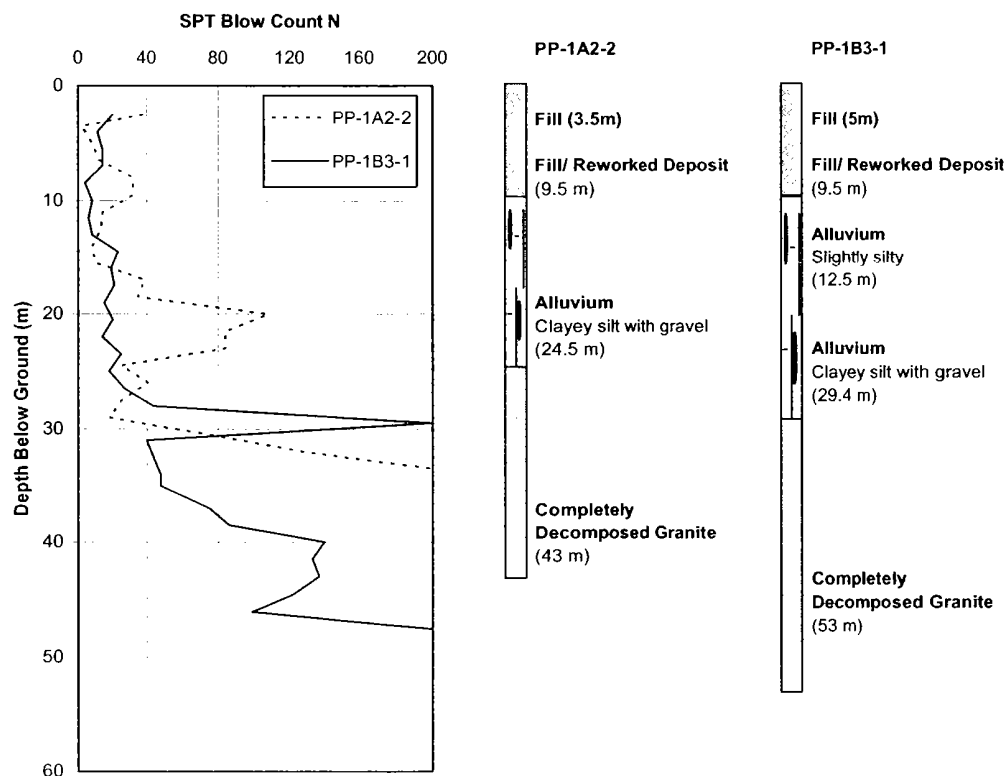


Fig. 5. Soil profiles and standard penetration test blow count N
PP-1A2-2 and PP-1B3-1 were final set based on the wave equation at the end of initial

driving. A Junttan 20S hydraulic hammer was used for PP-1A2-2 and a DKH1523 hydraulic hammer was used for PP-1B3-1. The final set table derived for PP-1B3-1 based on the wave equation is shown in Table 3. The final embedded lengths of the two piles were 55.8 m and 58.5 m, respectively. At the end of initial driving, the measured sets and elastic compressions were 32 mm/10 blows and 48 mm for PP-1A2-2, and 24 mm/10 blows and 55 mm for PP-1B3-1, respectively. The measured elastic compressions were close to the predicted values of 50 mm and 56 mm for the two piles.

Static load tests on the two test piles were conducted following the procedure outlined by PNAP66 (Buildings Department 2002). The measured load-settlement curves are shown in Fig. 6. The tests were carried out successfully to 3.3 P (P: the design load 3548 kN). Both the maximum displacements at 2.0P and 3.0P are smaller than the allowable displacements and the residual displacements after unloading from 2P are smaller than the allowable residual settlement of 6.7 mm. The static load tests confirm that the termination criterion based on the wave equation analysis at the end of initial driving is satisfactory.

Final setting information of 21 preliminary piles at two sites in Hong Kong was also used to verify the wave equation approach. All these piles had been subjected to static load tests. Based on their final setting information (namely, hammer information, pile information, final set and transferred energy), the capacity values of these piles can be predicted easily using GRLWEAP. Fig. 7 compares the pile capacity values measured from load tests and predicted from GRLWEAP. The predicted values are somewhat smaller than those obtained from the static load tests. The mean values of the ratio of the GRLWEAP capacity to the static load test capacity is 0.78 for cases at Site 1 and 0.74 for cases at Site 2. The comparisons in Fig. 7 do not suggest that the wave equation predictions are conservative. In fact, the pile capacity predicted by the wave equation approach is the capacity at the time of the final set test. The static load tests were usually performed 2 to 4 weeks after the final set test. During the period, the pile capacity may increase due to set-up effects.

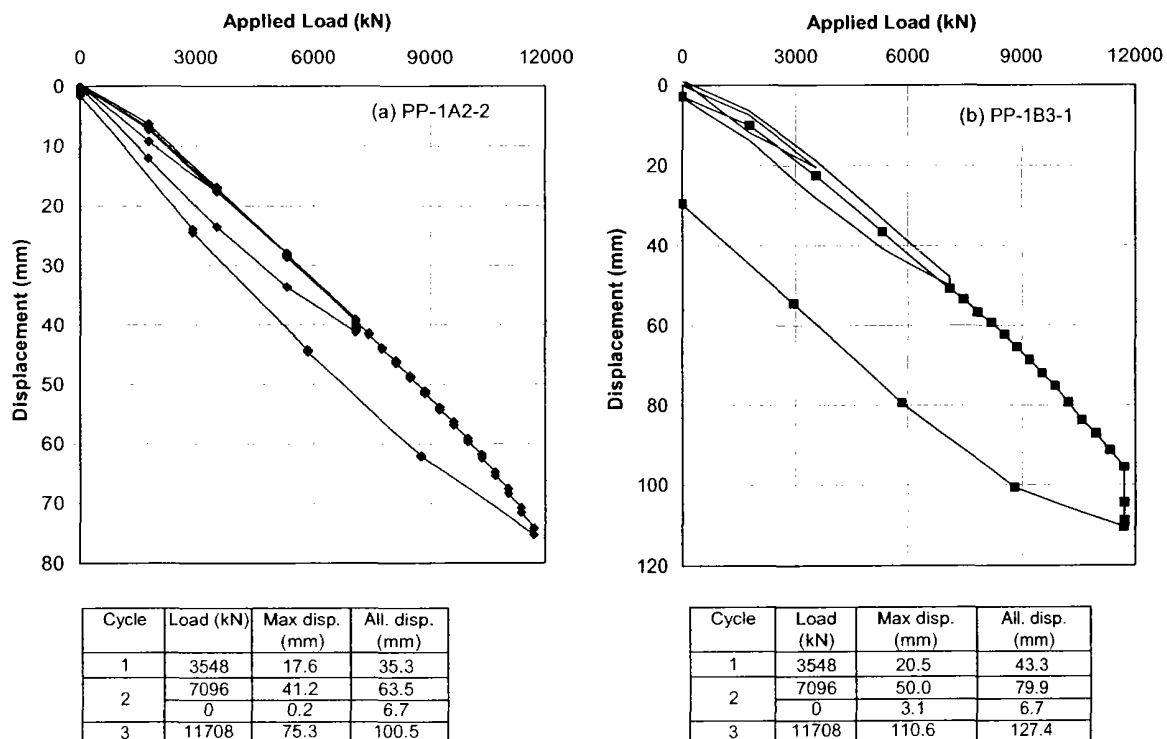


Fig. 6. Measured load-settlement curves from static load tests

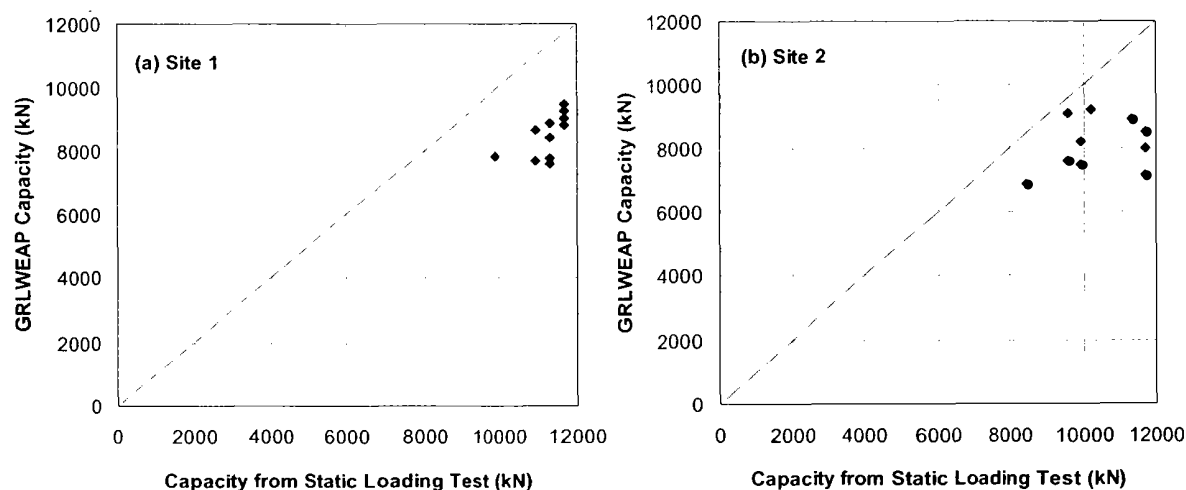


Fig. 7. Comparison between the pile capacity values measured from static load tests and predicted with GRLWEAP

CONCLUSIONS

The wave equation approach provides a means with which many aspects of pile driveability can be addressed. This paper outlines a methodology to develop final set tables for long driven piles using wave equation analyses. Soil parameters for wave equation analyses of driven piles in granitic soils were derived from a correlation study. With these parameters, final set tables for piles of different lengths can be developed. The final sets are not sensitive to slight changes in the hammer energy and the pile length. When the piles are long or the ground conditions are strong, the wave equation approach allows much larger final set values than the HKCA formula. Therefore, many pile integrity issues and construction delay problems can be minimised using the wave equation approach.

To verify the performance of piles final set using the wave equation approach, two preliminary piles, PP-1A2-2 and PP-1B3-1, were installed and load tested. Both the pile settlement values at twice the design load and the residual settlement values are well below the respective allowable values. Hence, the proposed methodology can provide a cost-effective yet safe solution to long piles. Besides these two preliminary piles, pile capacity values of 21 other preliminary piles final set by other methods were also predicted using the wave equation and the predictions were reasonably safe compared with those measured from static load tests on these preliminary piles.

REFERENCES

- Abe, S., and Thendean, G. (1996). "Hammer performance evaluations." Proc. 5th Int. Conf. on the Application of Stress Wave Theory to Piles, Orlando, FL.
- Buildings Department (2002). Pile Foundations-Practice Notes for Authorized Persons and Registered Structural Engineers PNAP66. Buildings Department, Hong Kong.
- Gammon, J.R.A. (1982). "Dynamic pile testing." *Hong Kong Engineer*, 10(8), 65-67.
- GEO (1996). *Pile Design and Construction*. GEO Publication No. 1/96, Geotechnical Engineering Office, Hong Kong.
- Goble, G. G. and Rausche, F. (1976). *Wave Equation Analysis of Pile Driving – WEAP Program*. Federal Highway Administration, U.S. Department of Transportation, Washington, D.C., Volumes I-IV.

- Goble Rausche Likins and Associates (2002). *GRLWEAP Wave Equation Manual*. Cleveland, Ohio.
- HKCA (1995). *Report on Hydraulic Hammers Performance*. The Hong Kong Construction Association, Hong Kong.
- Li, K. S., Chan, S. T., and Lam, J. (2003). "An alternative pile driving formula –A preliminary study." *Proc. Case Histories in Geotechnical Engineering in Hong Kong*. Hong Kong Institute of Engineers, Hong Kong, 179-189.
- Ng, F.H.Y., Cheung C.K.W., and Wong, K.F. (2000). "Application of dynamic and static design approach for piling in difficult ground conditions." *Proc. 19th Annual Seminar of the Geotechnical Division, HKIE*, 49-57.
- Poulos, H. G., and Davies, E. H. (1980). *Pile Foundation Analysis and Design*. John Wiley and Sons, New York.
- Pratt, M., Walsh, N. M., Arunachalam, S., Young S. and Sunderland P. (2000). "Deep foundations in Hong Kong." *Proc. Deep Foundation Institute, 25th Annual Meeting and 8th Int. Conf. and Exposition*, 69-83.
- Rausche, F., Goble, G., Likins, G. (1985). "Dynamic determination of pile capacity." *Journal of Geotechnical Engineering, ASCE*, 111(3), 367-383.
- Smith, E. A. L. (1960). "Pile-driving analysis by the wave equation." *Journal of the Soil Mechanics and Foundation Engineering Division, ASCE*, 86(4), 35-61.
- Thendean, G., Rausche, F., Svinkin, M. and Likins, G. (1996). "Wave equation correlation studies." *Proc. 5th Int. Conf. on the Application of Stress Wave Theory to Piles*, Orlando, FL, 144-162.
- Tomlinson, M.J. (1995). *Foundation Design and Construction, Sixth Edition*. Longman Scientific & Technical, Essex, England.
- US Department of Transportation (USDOT) (1996). *Design and Construction of Driven Pile Foundations – Workshop Manual – Volume II*. Washington D.C.
- Zhang, L.M., Chau, C.W., and Shek, M.P. (2003). "Bearing capacity of long driven piles using dynamic methods." *Proc. 12th Asian Regional Conf. on Soil Mech. & Geotech. Engg.*, Singapore, 685-688.

ACKNOWLEDGEMENTS

This research was funded by the Housing Authority Research Fund (Agreement No. B20030027, HKUST Project No. HARF02/03.EG05). A Research Management Team was formed to oversee the research activities. The team members include Dr Hon-Wah Pang (Director), Mr. Chan-Fai Pang (Secretary), and Messrs. Chi-On Chan, Martin W.T. Tsoi and C. F. Tang. Gammon Skanska Ltd. and China State Construction Engineering Company carried out the field tests on the two preliminary piles reported in the paper. The field works were coordinated by Mr. William K. H. Tang. The generous support of the three organisations and the technical input from the Management Team members and the coordinator are cordially acknowledged.

The authors would also like to thank the following individuals who played a key role in planning, coordinating and conducting the field tests: Messrs. Edmund Chung, W. K. Chan and Ben Wat of the Housing Authority, Mr. Dominic Lee of Gammon Skanska Ltd. and Messrs. Arthur So and C. K. Cheung of China State Construction Engineering Company.

NON-DESTRUCTIVE TESTS FOR DETERMINING THE LENGTH OF INSTALLED SOIL NAILS

W M Cheung¹ and D O K Lo¹

Abstract: Soil nailing has long been recognised as an effective and practical means for stabilising slopes and supporting excavations. It has been used extensively in slope improvement works in Hong Kong since the mid-1980s. However, once a soil nail has been installed, it is difficult to check its integrity, such as the length of the steel bar and the quality of cement grout sleeve. To enhance the quality control of soil nail installation works, a study has been carried out to identify and develop non-destructive means for determining the integrity of installed soil nails. The study comprises two phases: one on assessment of the length of the installed soil nail bars and the other on the integrity of the grout sleeves. This paper presents the findings of the first phase of the study. Various techniques including sonic echo, magnetometry, time domain reflectometry and electro-magnetic induction have been investigated both in the laboratory and in the field. The study concluded that some of the methods can be used to check the length of installed soil nail bars. The basic principles and accuracy of the techniques are discussed. Merits and limitations of these techniques are also presented.

Keywords: Non-destructive test, integrity, soil nail, nail length

INTRODUCTION

Soil nailing has been used extensively for slope improvement works in Hong Kong since the mid-1980s. To date more than 1,300 slopes have been retrofitted using soil nails through the government's Landslip Preventive Measures Programme. Similar to other geotechnical works, the performance of a soil-nailed slope is highly dependent on the construction quality of individual nails. Clear site supervision requirements are stipulated for soil nailing works to assure their quality. This paper describes the findings of a study on the identification and development of non-destructive means to determine the length of installed soil nails to supplement, where necessary, site supervision in respect of quality control of soil nailing works (Cheung, 2003).

TECHNIQUES FOR DETERMINING THE LENGTH OF INSTALLED SOIL NAILS

Five techniques have been identified for determining the length of installed soil nails. They are the Sonic Echo method, the Mise-a-la-Masse method, Magnetometry, the Electro-magnetic Induction method and Time Domain Reflectometry.

Sonic Echo (Low-strain Integrity) Method

The principle of this method is based on that of the low-strain pile integrity test (Turner, 1997). This method involves the examination of the generation, transmission and reception of an acoustic wave (i.e. stress wave or shock wave) in a steel nail bar or in a grout sleeve. An acoustic wave is usually excited by striking the bar head with a light hand-held hammer.

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An accelerometer is mounted either on the steel nail bar or on the top end of the grout sleeve. The passage of the acoustic wave along a steel nail bar and the grout embedding the bar will be affected by (i) the magnitude of the initial excitation force, (ii) the physical properties of the steel nail bar and grout sleeve, and (iii) the physical properties of the ground surrounding the nail. A major reflection of the wave is expected to occur at the nail end where there is a significant change in the physical property of the medium in which the wave is travelling. By analysing the reflected acoustic wave received either from the nail bar or the grout sleeve, one can estimate the location of the bar end. Figure 1(a) illustrates a theoretical response waveform of the test. In practice, wave energy will be dissipated as a result of damping effect. The magnitude of the successive reflected waveform received may progressively diminish as shown in Figure 1(b).

Site trials were carried out on 30 nails with length varying from 6 m to 16 m. Out of the 30 test nails, 6 nails (20% of the tested nails) have no recognisable reflections for interpretation, 8 nails (27%) have error in length estimation more than $\pm 10\%$ and 16 nails (53%) have error less than $\pm 10\%$.

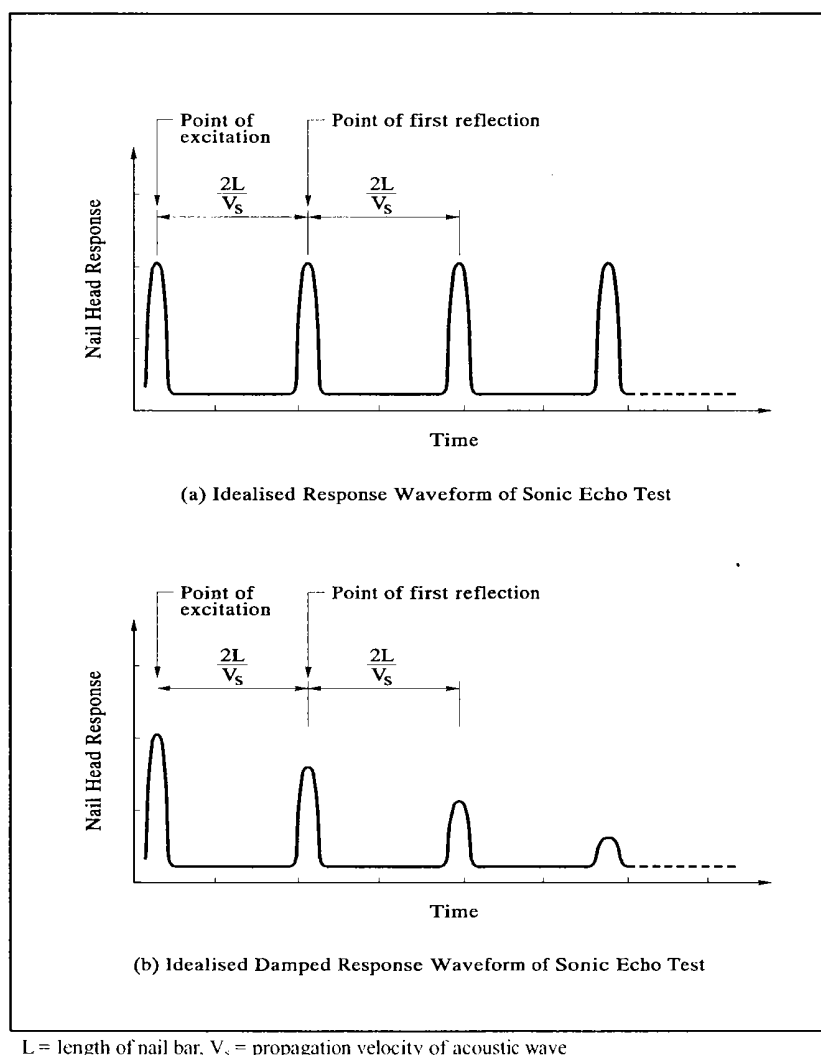


Figure 1 – Theoretical response waveform of sonic echo test

Mise-a-la-Masse Method

Mise-a-la-Masse method is mainly used in mining exploration (ASCE, 1998). The method is based on the principle that when an electrical energy is applied to two distant points on the ground through electrodes, an electric current, caused by the difference in potential, will flow between them. The electric current and potential distribution can be theoretically estimated for the ideal case where the ground between the two electrodes is homogeneous. If one of the electrodes is replaced by a steel soil nail and/or the ground is heterogeneous, there will be distortion in the electrical field which deviates from the ideal case. If the distribution of the distorted electrical potential is calibrated against steel soil nails of known lengths, the length of a test nail can be estimated theoretically based on the potential distribution around the nail at the same site. Figure 2 shows the theoretical spatial distribution of the electrical potential around a soil nail of a given length.

Trials were carried out at two test nails of 10 m and 12 m long. Although only two tests were carried out, the estimated nail lengths differ from the actual lengths by 8 m and 10 m respectively amounting to about 80% error. This indicates that the accuracy of this method is very low.

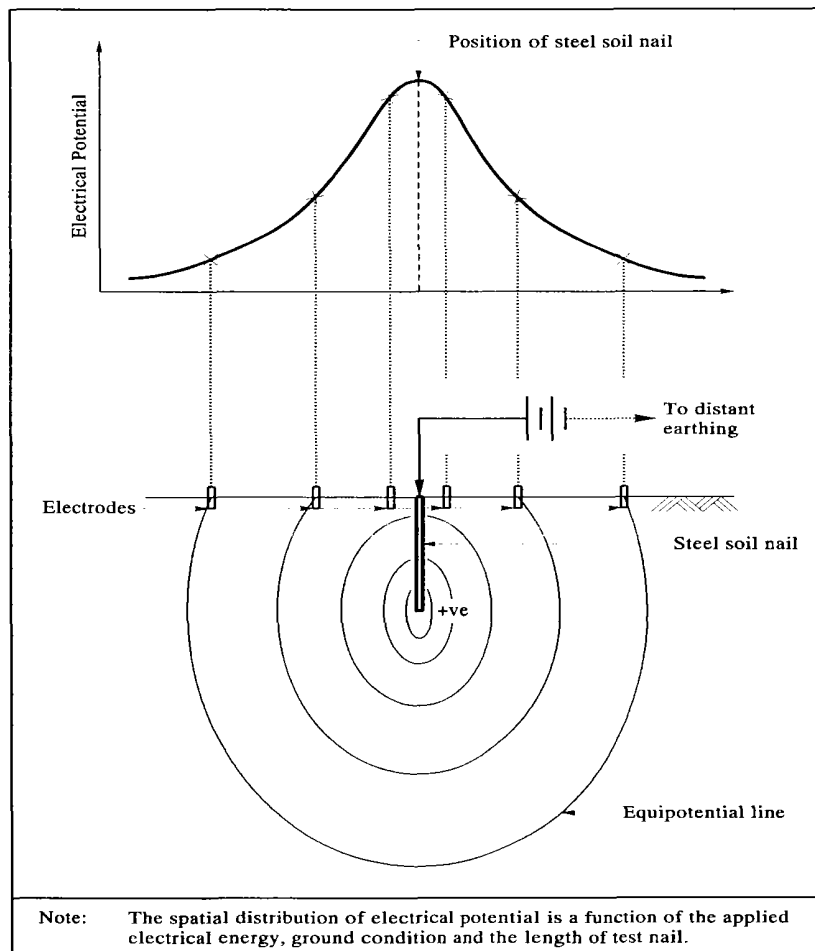


Figure 2 - Theoretical spatial distribution of the electrical potential around a soil nail of a given length.

Magnetometry

The Earth possesses a magnetic field caused by sources in its core. Ferrous materials, because of their high magnetic susceptibility, could lead to a substantial local distortion of the Earth's magnetic field. The principle of magnetometry is to measure the changes in this magnetic field strength due to the presence of ferrous materials, such as a steel bar, using a magnetometer (ASCE, 1998). A magnetometer is a probe of about 50 mm in diameter and 750 mm in length, which measures the Earth's magnetic field in three orthogonal directions. Figure 3 shows the theoretical magnetic vector profiles along a soil nail. By analysing the fluctuation of these magnetic vectors, the positions of the couplers and the bar ends can be identified.

In the field trial, probe holes of 100 mm diameter were drilled with alignment similar to that of the installed nails to be tested. The distance between the centre of the probe hole and that of the test nail was about 1 m to 1.5 m. The magnetometer was then inserted into the probe hole and measurements of the strength of the magnetic field were taken continuously as the probe advanced the hole. Tests were carried out on seven nails (actual nail length ranging from 6 m to 16 m), among which six have an error between -12% and 0%. The remaining test has an error of about -1.3 m for a 6 m nail (-22% error). This indicates that the accuracy of this method is relatively high, in particular for long nails.

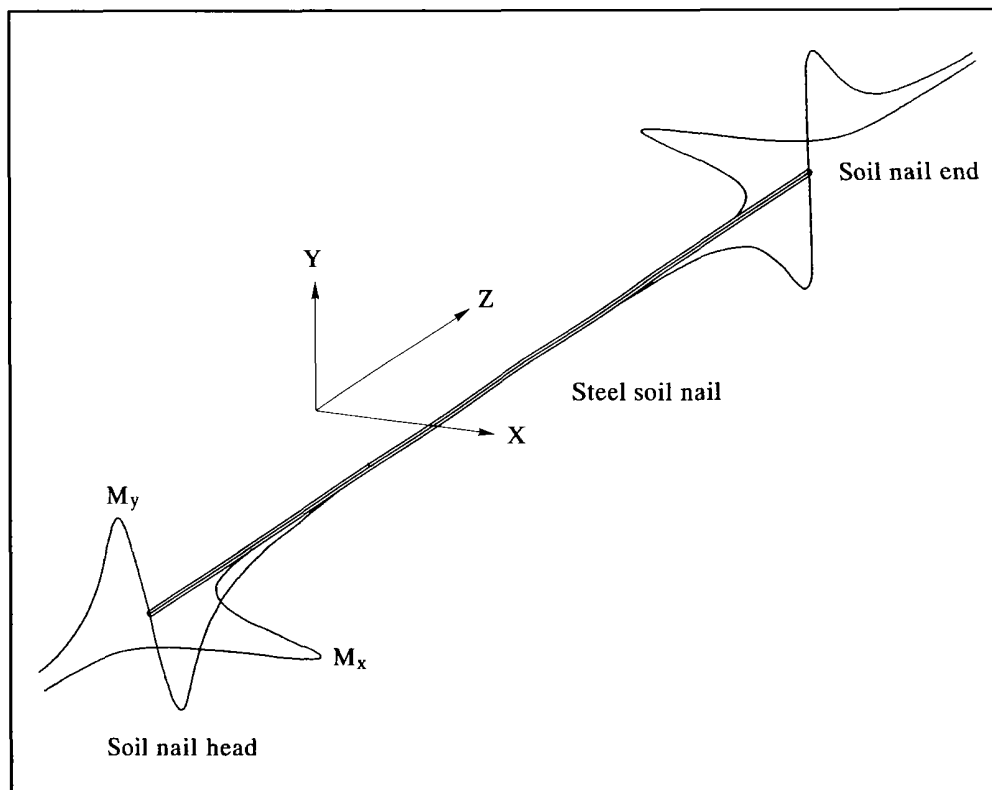


Figure 3 – Theoretical magnetic vector profiles along a soil nail

Time Domain Reflectometry

Time Domain Reflectometry (TDR) was first developed in the 1950s to detect faults in transmission cables (O'Connor & Dowding, 1999). When an electrical pulse is sent along a pair of closely placed conductors, changes in impedance along the pair would cause reflections. The characteristics of a reflection, such as its strength and polarity relative to the source signal, depend on the nature of the change in impedance. The time lapsed between the source and the reflection signals can be used to determine the location of the reflection if the transmission velocity of the signal is known. By pre-installing a copper wire alongside the nail bar, they form a pair of conductors. The propagation velocity of the pulse travelling along the wire and soil nail can be obtained from testing a nail with known length. Thereafter, one can estimate the length of a nail of unknown length by measuring the time of travel of the pulse from the nail head to its end together with the pulse propagation velocity obtained from the calibration nail. Figure 4 shows the typical waveform for a soil nail with a pre-installed wire.

Field trials were carried out on more than 200 nails with pre-installed wires using TDR cable fault detector. The mean of the errors in length estimation is less than 1% for nails with length ranging from 5 m to 20 m. All the errors fall within $\pm 15\%$. The high percentage error mainly relates to measurements in nails of short length (less than 6 m).

Field trial was also carried out using a proprietary TDR instrument on a smaller sample of nails without pre-installed wires. Fifteen nails of length varies from 6 m to 12 m have been tested, and the error falls within $\pm 10\%$. While the method is reasonably accurate, it requires experienced personnel to identify the reflection corresponding to the nail end. It was noted that the reflected signals in some cases were not as strong as that from one with a pre-installed wire. Sometimes, it may be even difficult to identify the reflected signal. As such, the reliability of this method without using a pre-installed wire is lower than that with a wire.

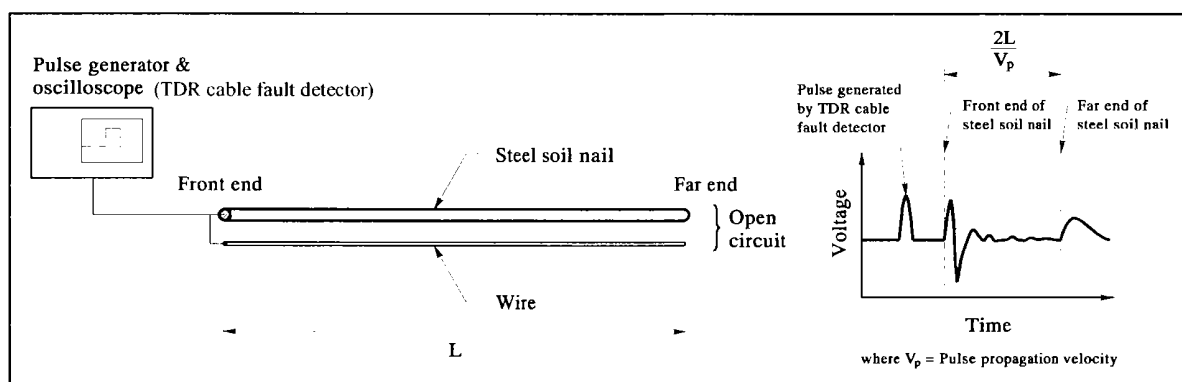


Figure 4 – Typical waveform from a soil nail with a pre-installed wire using TDR

Electro-magnetic Induction Method

Electro-magnetic induction is the influence by which an electric current produces magnetic polarity in certain bodies near or around which it passes. This method, which is developed by the Public Works Central Laboratory of Hong Kong, requires a pair of small diameter plastic tubes (about 10 mm) to be pre-installed along and at the opposite sides of the soil nail bar. These plastic tubes are extended beyond the end of the bar by about 50 mm.

Two probes, comprising steel solenoids wound with copper wire, are connected to plastic rods with length markings. During testing, the probes are inserted simultaneously, one into each of the pre-installed plastic tubes. A voltage is applied to the emitter probe. As illustrated in Figure 5, when the probes pass beyond the end of a steel soil nail (i.e. from the presence to the absence of a steel bar between the probes), there is a noticeable increase in the voltage induced in the receiver probe. Thus, by noting the markings of the plastic rod attached to the probes when a major change in the profile of the voltage induced in the receiver probe, the end of a steel soil nail can be identified.

Field trials were carried out at two sites. The test results of all the eleven nails of length varying from 12 m to 20 m indicate that the method is reliable and has the highest accuracy (error within $\pm 1\%$) amongst the five methods in this study.

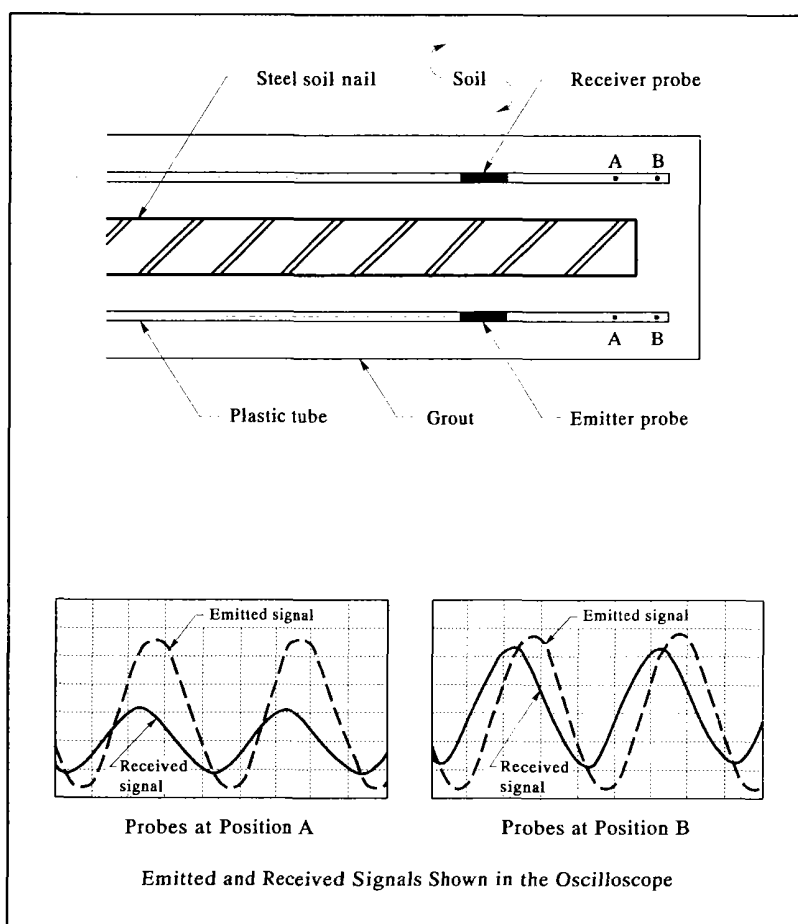


Figure 5 – Working principle of the electro-magnetic induction method

MERITS AND LIMITATIONS

The merits and limitations of the five techniques are summarised in Table 1. It is evident that the Sonic Echo and the Mise-a-la-Masse methods have the lowest accuracy among the five methods. The Electro-magnetic Induction method is the most accurate and reliable among the studied methods. However, the need to pre-install two plastic tubes and the uncertain effects of the tubes on the integrity of grout sleeve may render this method less attractive at this stage. Magnetometry and Time Domain Reflectometry are considered

accurate. In summary, the Electro-magnetic Induction method, Magnetometry, and Time Domain Reflectometry in particular for a nail with a pre-installed wire, are considered sufficiently accurate and reliable for practical use. Apart from Time Domain Reflectometry on nails with pre-installed wires, only a relatively limited number of measurements have been carried out for each method. It is recommended that more extensive tests be conducted to affirm and quantify the accuracy of these methods. It is worth noting that the limitations of various techniques corresponded to that noted at the time of the study. With the advent of technology, it is envisaged that some of these limitations could soon be overcome.

	Sonic Echo Method	Mise-a-la-Masse Method	Magnetometry	Time Domain Reflectometry		Electro-magnetic Induction Method
				With wire	Without wire	
Merits	- quick - instrument readily available in local market - feasible for all soil nailed slopes	- feasible for all soil nailed slope	- quick - both the locations of couplers and bar end can be identified - feasible for all soil nailed slope	- quick - experienced personnel for result interpretation is not required - instrument readily available in local market	- quick - feasible for all soil nailed slope	- quick - experienced personnel for result interpretation is not required - instrument readily available in local market
Limitations	- reliability subject to attenuation and partial reflection - require experienced personnel to interpret test results - preferable to have calibration with nails of known lengths	- time consuming - reliability subject to high uncertainties of ground resistivity and extraneous electrical currents - require experienced personnel to interpret test results - instrument may not be readily available in local market - require calibration with nails of known lengths	- probe hole is required - results near slope surface subject to interference by overhead power lines, radio broadcasting stations, etc. - require experienced personnel to interpret test results	- pre-installation of wire is required - not feasible for soil nailed slopes without provision of wire - require calibration with nail of known length	- require experienced personnel to interpret test results - reflection may not be as strong as that obtained from a nail with a pre-installed wire - require calibration with nail of known length - instrument not readily available in local market	- pre-installation of plastic tubes is required - effect of plastic tubes on the integrity of soil nail is unknown if left open
Accuracy	Low	Very Low	High	High	High	Very High
Reliability	Moderate	Low	High	High	Moderate	High

Table 1 – Merits and limitations of different methods

CONCLUSIONS

A study has been carried out to identify and develop non-destructive means for determining the length of installed soil nails. From the trial use of various techniques to determine the lengths of soil nails, it is found that Magnetometry, the Electro-magnetic Induction method, and Time Domain Reflectometry are both reliable and sufficiently accurate for practical purposes. It is recommended that more extensive tests be conducted to affirm and quantify the accuracy of these methods.

REFERENCES

- ASCE (1998). *Geophysical Exploration for Engineering and Environmental Investigations – Technical Engineering and Design Guides as Adapted from the US Army Corps of Engineers, No. 23*. American Society of Civil Engineers, 204 p.
- Cheung, W.M. (2003). *Non-destructive tests for determining the lengths of installed steel soil nails*. GEO Report No. 133, Geotechnical Engineering Office, Hong Kong, 54 p.
- O'Connor, K.M. and Dowding, C.H. (1999). *GeoMeasurements by Pulsing TDR Cables and Probes*. CRC Press, 394 p.
- Turner, M.J. (1997). *Integrity Testing in Piling Practice*. Report No. 144, Construction Industry Research and Information Association (CIRIA), 336 p.

ACKNOWLEDGEMENTS

This paper is published with the permission of the Head of the Geotechnical Engineering Office and the Director of Civil Engineering, the Government of the Hong Kong Special Administrative Region.

UNDERGROUND AND FOUNDATION WORKS MONITORING USING SURFACE WAVE TIME DOMAIN REFLECTOMETRY TECHNIQUE

Terrence Tang¹

Abstract: A new measurement technique has been developed to monitor the length and integrity of underground metal structures; such as soil nails, reinforced concrete piles (bored piles), mini-piles and H piles etc. The same technique can be used to monitor land mass movement, underground water table, movements in foundation structures and corrosion in reinforced steel bars etc. The method is based on Surface Wave - Time Domain Reflectometry (SW-TDR).

INTRODUCTION

Soil nails, H-piles, Mini-piles and Reinforced Concrete Piles (bored piles) are common underground metal structures in geotechnical or civil engineering works. The functions of these structures are to maintain the stability of slopes (for the case of soil nails) or to transmit compression load or sometimes tension onto/from the ground (for case of pilings). Quality assurance for these structures will be very important. However, as the structures are completely embedded underneath the ground after construction, any quality monitoring on the works will be difficult. The measurements have to be non-destructive and in-situ. Surface Wave - Time Domain Reflectometry (SW-TDR) technique would be recommended for this type of works monitoring.

Principle of Surface Wave - Time Domain Reflectometry (SW-TDR)

This is based on Time Domain Reflectometry, using Surface Electro-magnetic Waves on single conductor as the probing waves. The method works on the same principle as radar. High frequency step-like Surface Electromagnetic Waves are generated and let transmitted onto a metal conductor of the structure under test. The metal conductor can be the steel reinforcement in a soil nail, the reinforcement cage in a bored pile or I-beam in an H-pile etc. The metal conductor acts as a waveguide for the Surface Electromagnetic Waves. The waves propagate in the concrete/grout region near and along the metal conductor. The materials properties surrounding the metal conductor will determine the velocity of the probing waves. For general information on Surface Waves, readers are referred to papers by Sommerfeld [1], Zenneck [2], Goubau [3] and Ling et al [6] or text books by Stratton [4] and Jordan et al [5]. Propagation of Surface Wave along a conductor is diagrammatically shown in Figure 1.

When Surface Waves reach places where the local electrical impedance differs from the characteristic electrical impedance of the metal conductor under test, part or all of waves will be reflected back. The polarity of return waves will depend on the nature of impedance variation. The change of electrical impedance can be caused, for instance, upon the arrival of the waves at the conductor end or the change of material properties surrounding the metal conductor, (such as in the presence of voids or conductor in direct contact with surrounding

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soil etc.) SW-TDR records the entire wave signatures during the wave transmission and reflection process. By measuring the time of reflections and analyzing the return wave profile, many parameters such as length, materials integrity and conditions around the conductors can be deduced.

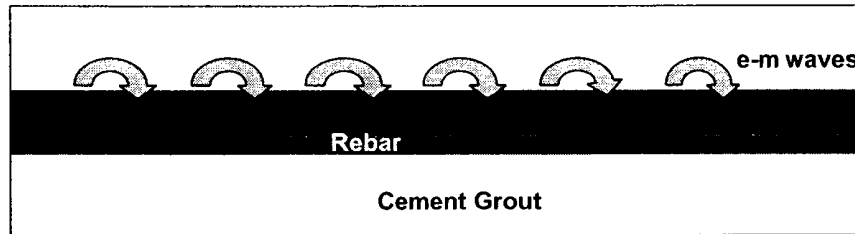


Figure 1 Surface Electromagnetic Waves propagating along a conductor

For cases such as soil nails, the electrical impedance, Z , relates to its inductance per unit length, L , and capacitance per unit length, C . The relationship can be approximated to:

$$Z \sim \sqrt{L/C}$$

The propagation velocity of the Surface Wave, v , is given by

$$v \sim \frac{1}{\sqrt{LC}}$$

With the presence of void in cement/concrete mix around the metal structure, the local capacitance value will be decreased, leading to an increment of electrical impedance. This will result in positive wave reflection. The coefficient of wave reflection, Γ , is given by:

$$\Gamma = \frac{Z_l - Z_0}{Z_l + Z_0}$$

where Z_l is the local electrical impedance value and Z_0 is the characteristic electrical impedance of the entire metal conductor.

On the contrary, when the metal object is in direct contact with soil or rock materials which has a higher electrical permittivity than the cement grouting or concrete mix, the local capacitance value will be increased. This will lead to the lowering of local electrical impedance and hence negative wave reflection for the Surface Waves.

Upon the waves reaching the metal conductor's ending, all the waves will be reflected back. In comparing to reflections arising from impedance anomalies along the metal conductor, the reflection from the ending position will be, therefore, largest. The polarity of wave reflection from the ending position will again depend on the terminating conditions of the metal object, in a similar manner as above. Figure 2 shows the theoretical wave profile measured at the nail head for a soil nail with voided and soiled region along its grout core.

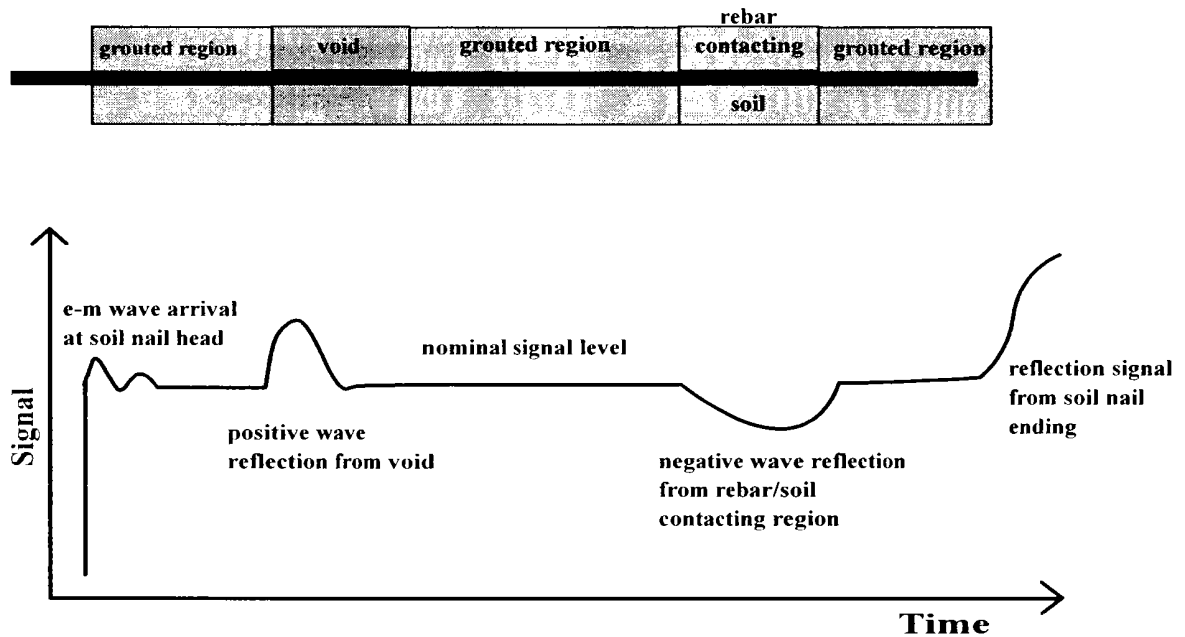


Figure 2: Soil nail with anomalies along rebar (above) and corresponding TDR wave profile (below)

The wave profile is a scan of impedance variation along the metal conductor. Since SW-TDR measurement is taken at the signal sending end, the time to reflection from any impedance anomalies will correspond to 2 distances travelled.

Examples of SW-TDR Wave Profiles

Following wave profiles were examples of real case measurements on some common underground metal structures. These include soil nails (figure 3a), soil nail with caved-in region (figure 3b), bored pile (figure 3c) and mini-pile (figure 3d). In all these examples, the times at which the Surface Wave entered the metal structure and reflected from the conductor's ending were marked by arrow indicators.

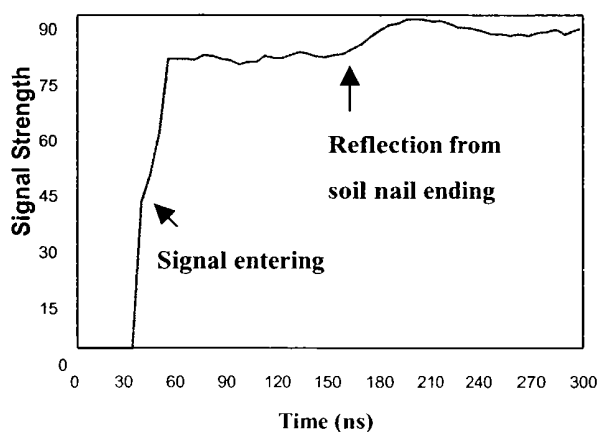


Figure 3a Wave profile from a soil nail

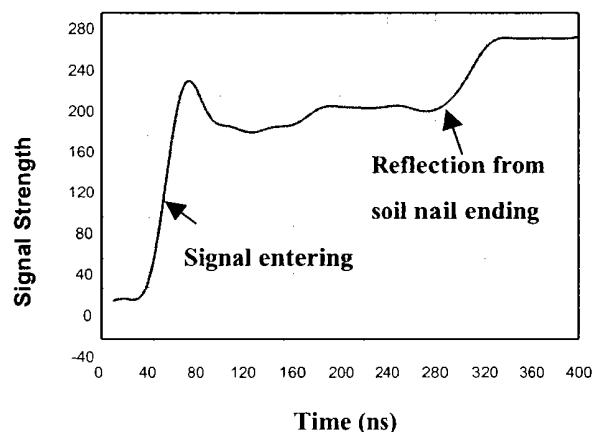


Figure 3b Wave profile from a soil nail with 50% caved-in region

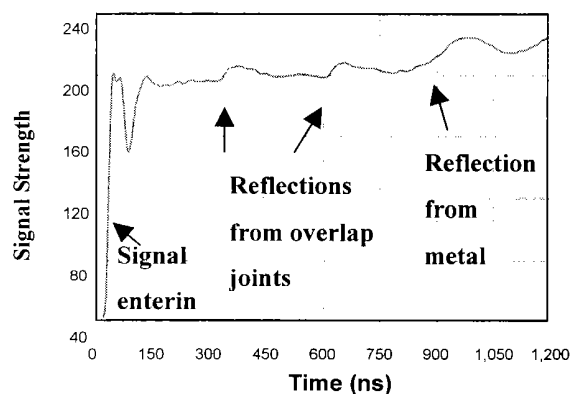


Figure 3c Wave profile from a bored pile

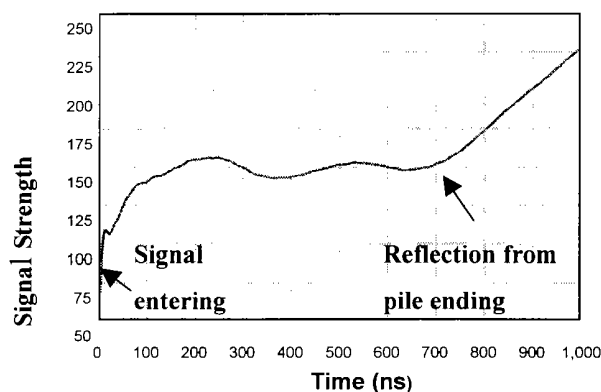


Figure 3d Wave profile from a mini-pile

GEOLOGICAL CONDITIONS AROUND METAL STRUCTURES

Length of Metal Structure

Surface Electromagnetic Waves propagate long distance with little attenuation. Couplers or clamps between conductors present no problem for the transmission of electromagnetic waves. The total length of underground metal structures can be up to several hundred meters.

Different Soil Conditions

Since Surface Electromagnetic Waves propagate in the concrete/grout region near and along the metal conductor surface, any variance of soil moisture content or type of soils surrounding the underground structure will have no effect on the propagation of the waves.

ACCURACY OF SW-TDR

From numerous measurements on soil nails, bored piles, mini-piles and H-piles etc., at different sites under different conditions and at different times, the overall accuracy of SW-TDR technique on length measurements was found to be within 6% of the true metal structure length. The soil nail length measurement data by Time Domain Reflectometry method stated in GEO report 133 were originated from the present author's previous work measurements.

SOIL NAIL LENGTH TESTING

Recently, the Geotechnical Engineering Office of Civil Engineering Department has recommended a pre-installation of an electrical wire along the soil nail steel reinforcement and that the length measurement of the soil nail after work completion be measured with a TDR cable tester. In comparing to the present SW-TDR method where Surface Wave propagates directly on the surface of the steel reinforcement with no requirement for

additional electrical wire, the wave reflection amplitude measured by a TDR cable tester in rebar-wire installation case, would be larger. However, there are many situations could counteract the merit of adding an electrical wire, the result of which would lead to totally inaccurate measurement conclusions, such as in cases:

- i) If a wire pair is installed along the steel reinforcement instead of a single wire, with one wire attached to the rebar whilst the other wire exposed outside the nail head as specified, the time duration to the ending reflection will be affected or controlled by the wire pair length. There still will be a strong signal reflection from the wire pair even in case when there is actually no rebar in the interior region at all. The measured signal could be totally non-related to the existence of steel reinforcement.
- ii) Various separation distances between wire and rebar, and conditions of wire along the passage will cause different electromagnetic propagation velocity. This could lead to different reflection duration for same length of steel reinforcement.
- iii) Even in case where there is no separation variation between the wire and steel reinforcement, the propagation velocity of e-m waves (TEM wave mode) in the rebar - wire arrangement case will still vary non-linearly with soil nail length; with higher velocity for a shorter soil nail and slower velocity for a longer nail situation. Thus, if e-m wave velocity is calibrated at site using a particular soil nail, a TDR cable tester would indicate soil nails to be excessively long (than their true values) for those soil nails which are longer than the calibration soil nail. Conversely, the measurement results will indicate a shortage of length for those soil nails which are shorter than the calibration soil nail. The results are inaccurate.

OTHER POTENTIAL APPLICATION OF SW-TDR TECHNIQUE

Slip Plane Monitoring

A sensor tube is installed in a grouted hole. Land mass movement causes localized tube shearing at the slip plane. This will result a localized electrical impedance variation at the sheared place, causing signal reflection of the probing Surface Electromagnetic Waves. Comparisons of return signals taken at different times will enable the monitoring of land slip process and the development of a pre-slip warning system, as indicated in figure 4.

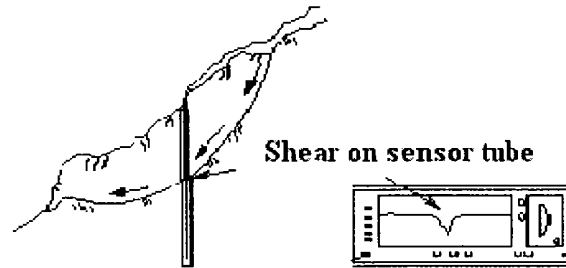


Figure 4 Sensor tube installed in a grouted hole

Underground Water Table Monitoring

For this type of measurement, ground water is used as the separation medium between 2 coaxial conductors. Variation of soil water pressure causes changes of water level in the sensor tube, which will be detected from reflection of Surface Electromagnetic Waves. The soil water pressure profiles at different depths can be deduced from a bundle of such sensor tubes. Figure 5 shows wave profiles for a sensor cable filled with water to different depths.

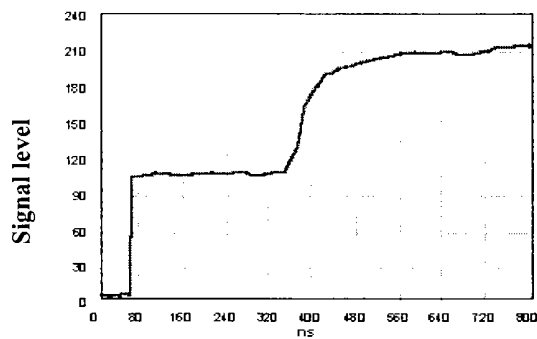


Figure 5a 30m sensor tube under totally dry condition

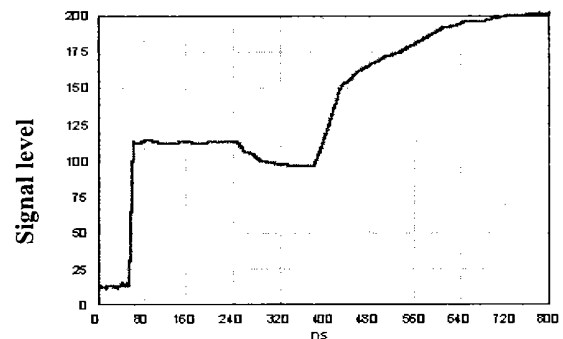


Figure 5b 30m sensor tube with 10m water column at bottom

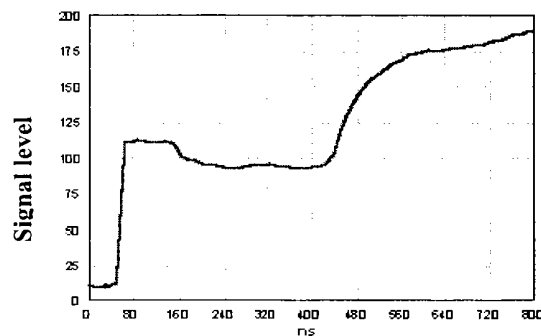


Figure 5c 30m sensor tube with 20m water column at bottom

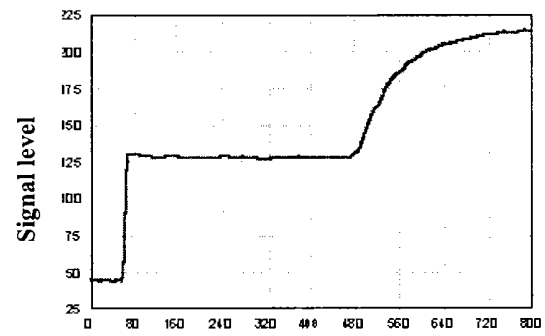


Figure 5d 30m sensor tube totally filled with water

Detection of leaks in metallic water pipes

A metallic water pipe with concrete/bitumen protective layer will behave electrical similar to a soil nail. When leaks occur in the pipe, this will make the pipe more electrically connected to the surrounding soils via the leaking water. The electrical impedance at the leaking place will be different from the normal characteristic electrical impedance of the pipe. Upon the arrival of Surface Waves at the leaking places, the local impedance variation will lead to a negative reflection for the probing waves. Location of leaks can be deduced from the time of wave reflections. Figure 6 shows detection of a leaking pipe.

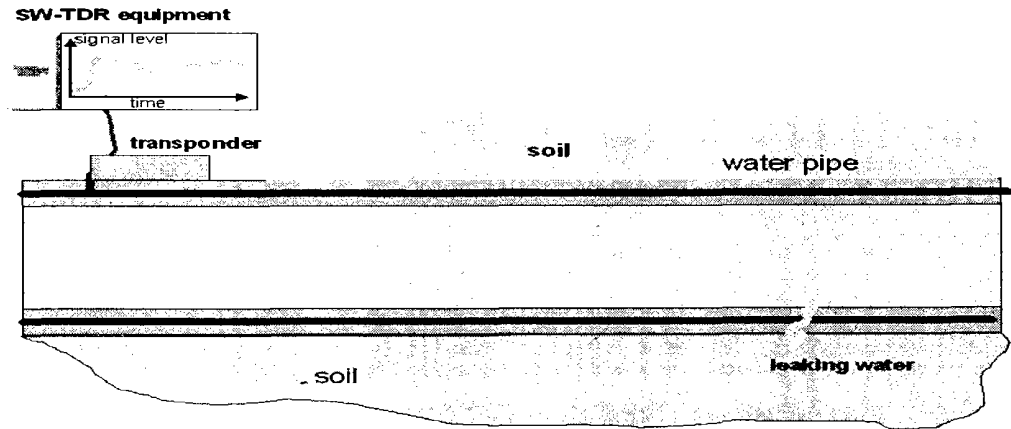


Figure 6 Detection of leaking in metallic water pipe

Detection of Corrosion

For any corrosion on the rebar surface, the corrosion region will cause a resistive effect to the Surface Waves (as indicated in Figure 6), resulting in higher return signal from the corroded region. With more corrosion on the rebar surface, more wave reflection will occur, as shown in Figure 8.

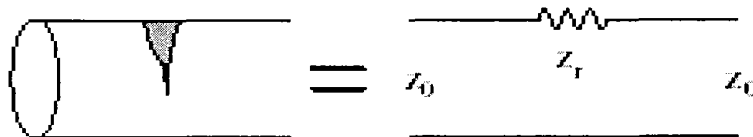


Figure 7 Surface corrosion on steel bar with equivalent circuit

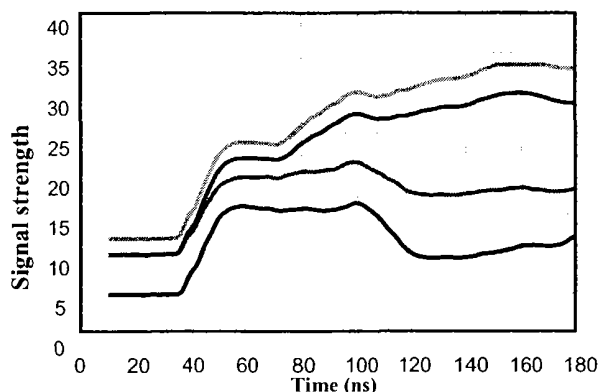


Figure 8 Stronger wave reflections from same location with more surface corrosion.

The wave profiles were overlapped for easy comparison.

LIMITATIONS OF SW-TDR

Surface Wave Time Domain Reflectometry is a new technique which may not be familiar with by most civil or geotechnical engineers. For full extraction of information on the measured TDR wave diagrams, wave interpretations are required for each SW-TDR measurement. This will require experienced personnel for wave interpretation. However, to help wave recognition and calculation of anomalies for less experienced personnel, a set of reference SW-TDR wave diagrams from known defective situation would be recommended. For defects of same nature, the wave profiles should be similar to that of reference guide, although the time scale for each case may be different because of sizes of anomalies. The sizes of anomalies along the metal conductor can be calculated using the calibrated Surface Wave velocity at site. As Surface Wave propagates typically in an order of speed of light, this requires the instrument to be very accurate and fast. Modern electronics would give SW-TDR technique a distance resolution of few centimeters, regardless of length of the metal structures.

CONCLUSION

SW-TDR technique can be used to monitor the length and integrity of underground metal structures or used as a tool for continuous monitoring of on-going underground activities, such as land mass movement, and water table etc.

REFERENCES

- 1 Sommerfeld, A. "Transmission of electrodynamic waves along a cylindrical conductor" *Annalen der Physik und Chemie* Vol. 67, 233-20-, 1899
- 2 Zenneck, J., "On the propagation of plane electromagnetic waves along a planar conductor surface and its relationship to wireless telegraphy", *Annalen der Physik*, Series 4, Vol 23, 848-866, 1907
- 3 Goubau, G., "Surface waves and their application to transmission lines", *J. Applied Physics*, Vol 21, 1119-1128, 1950
- 4 Stratton, J.A., *Electromagnetic Theory*, McGraw Hill, New York, 1941
- 5 Jordan E., and Balmain K., *Electromagnetic waves & Radiating Systems*, Prentice Hall Ltd. 1976
- 6 Ling R.T., Scholler J.D and Ufimtsev P Y., "The propagation and excitation of surface waves in an absorbing layer", *Progress in Electromagnetic Research*, PIER 19, 49-91, 1998

ACKNOWLEDGEMENTS

The comments from the Organizing Committee of the 2003 HKIE Geotechnical Division Annual Seminar are gratefully acknowledged.

NEW CAPABILITY AND OPPORTUNITIES FOR USING DIGITAL TECHNOLOGY IN GEOTECHNICAL ENGINEERING

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Abstract: Digital technology offers an opportunity for advancing geotechnical practice and enhancing the capability and efficiency of geotechnical work. This paper describes a number of novel digital technologies, which have notable potential for geotechnical application in Hong Kong. These include digital photogrammetry, Geographic Information System (GIS), Interferometric Synthetic Aperture Radar (InSAR) and Light Detection and Ranging (LIDAR). Their principles, capability, current application and future potential are presented. These technologies have been identified by the HKIE Interest Group on Application of Innovative Technology in Geotechnical Engineering as novel techniques that warrant strategic attention.

INTRODUCTION

Significant advances have been made in digital and related novel technologies over the years. The technologies have become more readily accessible, with improved capability and reduced cost. Hong Kong is one of the parties taking the lead in applying novel technologies to geotechnical work (Wong 2001), and some of these applications are recognized as the state-of-the-art that has brought about enhanced geotechnical capability (SSTRB 2003 & 2004).

In recognition of the importance of novel technologies and their potential application, an Interest Group on Application Innovative Technology in Geotechnical Engineering was set up by the Geotechnical Division of the Hong Kong Institution of Engineers (HKIE) in mid-2003, with members from academic institutions, Geotechnical Engineering Office (GEO), and geotechnical practitioners in Hong Kong. The Group is tasked with developing knowledge and promoting geotechnical application of innovative technologies, which would help the geotechnical profession to meet new challenges in Hong Kong and to lead the professional excellence in the region.

The Group has identified the following eight strategic subjects for examination and promulgation of their geotechnical application:

- (1) construction control and technology;
- (2) digital photogrammetry;
- (3) Geographic Information System (GIS);
- (4) Global Positioning System (GPS);
- (5) instrumentation;

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- (6) Interferometric Synthetic Aperture Radar (InSAR);
- (7) Light Detection and Ranging (LIDAR); and
- (8) new geotechnical subjects, including environmental geotechnology, natural terrain landslide hazard and quantitative risk assessment.

Items (2), (3), (6) and (7) above directly involve the use of novel digital technologies. Their technological principles, capability, current applications and future potential are summarized in this paper. Item (4) above is partly related to GIS and LIDAR application, and is also described in such context in this paper. Three 'case report' papers, which supplement this theme paper, are included in the proceedings of this Seminar. These papers give a detailed account of selected application cases, which include ground movement detection using InSAR (Ding et al. 2004a), mobile GIS mapping (Ng et al. 2004) and boulder identification using image analysis techniques (Shi et al. 2004).

DIGITAL PHOTOGRAMMETRY

Hong Kong has a comprehensive collection of historical aerial photographs that are accessible to the public. These aerial photographs have long been an important source of information for geotechnical studies. Conventionally, aerial photograph interpretation (API) and photogrammetric analysis are carried out by skilled personnel using stereoscope and stereo-plotter (Figure 1). The work can now be undertaken by digital means via digital photogrammetry, with improved efficiency, resolution and analytical capability. It is carried out by digitizing a pair of aerial photographs with the use of a high precision and high resolution scanner, processing the digitized data together with the available control point data by an advanced digital photogrammetry algorithm, and displaying the stereo-images and processed data on a computer monitor (Figure 2). Standard, off-the-shelf hardware and software packages are available for digital photogrammetric analysis and presentation of the results. The set-up cost is around HK\$ 0.5 million. After setting up, the digital photogrammetry system is neither costly nor difficult to operate, particularly for personnel with API or surveying experience.

Digital photogrammetry has a number of

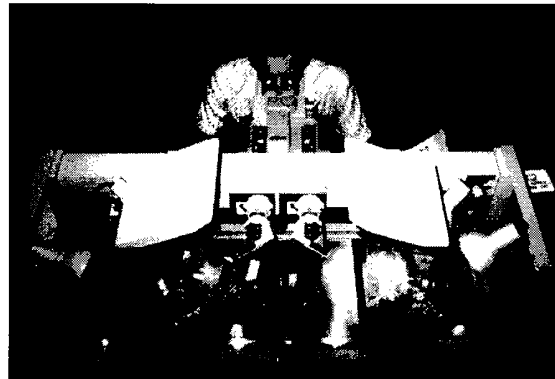


Figure 1 – Stereoscope used in conventional aerial photograph interpretation

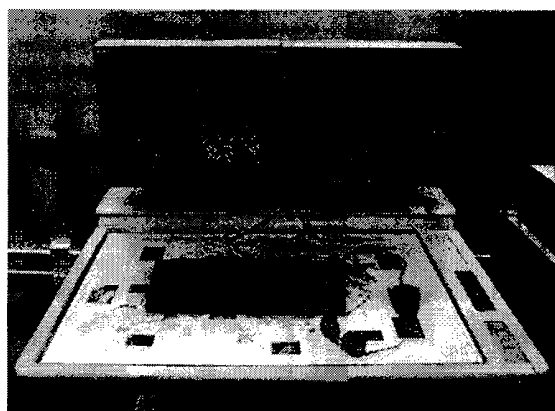


Figure 2 – Digital photogrammetry workstation (Note: stereo view displayed on computer monitors)

notable applications to geotechnical work. These include:

(1) Stereo visualization and API

With the ability to display 3-D stereo images on a computer monitor, stereo views of the present and past conditions of a site can be generated by the available historical aerial photographs and stereo visualization is easy, even for people without API training. Furthermore, it is feasible for API to be done and evaluated jointly by a team (Figure 3), which greatly facilitates communication and discussion. Good resolution can be achieved, e.g. up to about 0.1 m for vertical aerial photographs taken at 4,000 feet. Remote sensing and image analysis techniques can be applied to the digitized images for terrain evaluation and feature identification.

This provides an objective, consistent and less operator-dependent means of analyzing the aerial photographs, which supplements conventional API using stereoscopes.



Figure 3 – Aerial photograph interpretation by a team using digital photogrammetric technology

(2) Surveying and measurement

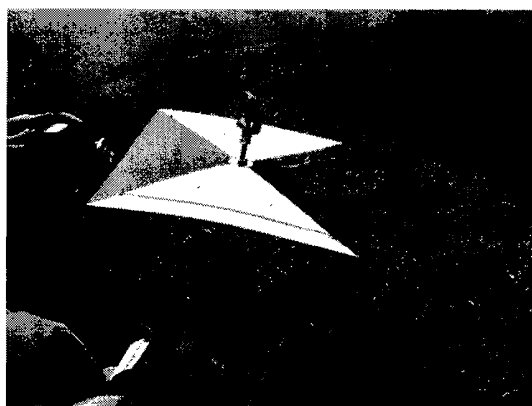
Comprehensive data on topography and feature dimensions can be derived from a pair of aerial photographs by digital photogrammetry, without the need for detailed land surveying work. A spatial accuracy of about 0.5 m to 1 m is achievable in Hong Kong, and this can be further improved with the use of low-flight photographs and additional ground control points. This accuracy can meet the need of topographic survey and measurement for many routine geotechnical applications, e.g. slope design. In this respect, digital photogrammetry offers an efficient and inexpensive means of remote surveying and measurement. This is particularly useful for geotechnical work that covers a large area, e.g. assessment of natural terrain hazards, and for circumstances where field measurement is dangerous or not possible, e.g. in areas with access problem or unstable slopes.

(3) Movement monitoring

Movement monitoring normally requires a higher degree of accuracy than that commonly required for visualisation and measurement purposes. Hence, digital photogrammetry using conventional aerial photographs (4,000 feet or above) would have limited use in movement monitoring, unless where the movement to be measured is large. Figure 4 shows an example of such application undertaken by the GEO at a large scale landslide site at Ma On Shan. Use of low-flight aerial photographs (e.g. taken at 500 feet) or terrestrial photogrammetry can give a better accuracy (e.g. Hanson and Lichti 2002; Li and Mo 2003), and there is scope for technological development in this area.



(a) Ma On Shan landslide site



(b) Marker installed on site for use as reference point in digital photogrammetric analysis

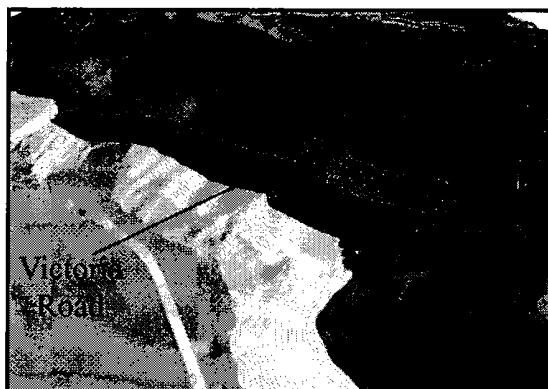
Figure 4 – Slope movement monitoring at Ma On Shan by digital photogrammetry

(4) Compilation of digital terrain model (DTM)

Digital terrain model (DTM) is a specific surveying and measurement product of digital photogrammetry that has important 3-D GIS and virtual reality application. Hong Kong has a full set of 1:1,000 Land Information Centre topographic maps, which can be used to generate DTM. Digital photogrammetry can provide supplementary DTM for areas where elevation data (i.e. contour lines and spot heights) are missing, or where more accurate DTM is required, or where the DTM of the site conditions in the past is required. However, in generating DTM by digital photogrammetry, due account should be taken to map the ground surface in area where vegetation is present. The GEO is about to complete a project to upgrade the current HK-wide 5m-grid DTM to 2 m-grid. Improvement has also been made in the provision for breaklines, such as drainage lines, roads and slopes, to remove artifacts and enhance the accuracy of the DTM in modeling ground features (Figure 5). This DTM will enable a more realistic representation of the terrain components and morphology, as well as improve the accuracy of geotechnical and other spatial assessments that require the use of 3-D topographic data.



(a) 5-m grid DTM



(b) 2-m grid DTM with enhancement (Note: roads and man-made slopes are correctly modelled in the DTM)

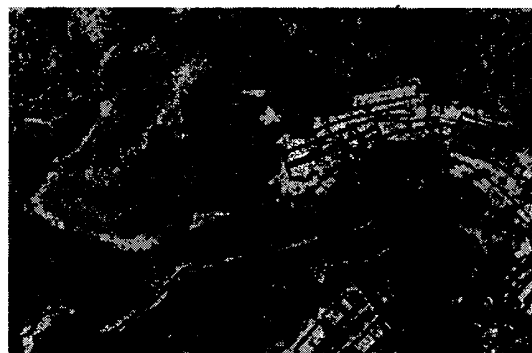
Figure 5 – Enhanced 2 m-grid digital terrain model (DTM)

(5) Production of ortho-rectified images

By application of digital photogrammetric techniques, conventional aerial photographs can be converted into ortho-rectified images. Such images are true to scale and position-accurate, which can supplement or even replace survey plans (Figure 6). Ortho-rectified images contain rich visual details of the ground at the time of taking the aerial photographs and are most suited for use in field reconnaissance and mapping. A list of the HK-wide ortho-rectified images available in the GEO is given in Table 1. As ortho-rectified images are in digital and geo-referenced format, they can be integrated into a GIS together with other spatial data for a range of novel GIS and remote-sensing applications. These include: geotechnical field mapping, GIS data mining, feature recognition and extraction, change detection and monitoring, and virtual reality. A notable application to mapping of boulders using image processing techniques is reported in Shi et al. (2004).



(a) Ordinary aerial photograph
(Note: not to scale and position-inaccurate)



(b) Ortho-rectified image
(Note: true to scale and position-accurate)

Figure 6 – Conversion of ordinary aerial photograph to ortho-rectified image

Table 1 – List of HKSAR-wide Ortho-rectified Images Held in GEO

Type of Image	Year	Color	Resolution
Satellite images (LandSat & SPOT 40 ⁺ sets)	1987 onward	Color	10 m +
Low-flight aerial photographs	1963 ⁽¹⁾	Black and White	0.1 - 0.5 m
	1973/74 ⁽¹⁾	Black and White	0.5 m
	1982 ⁽¹⁾	Black and White	0.5 m
	1993 ⁽¹⁾	Black and White	1.0 m
	2000 ⁽²⁾	Color	1.0 m
	2001 ⁽²⁾	Color	1.0 m
	2002/3 ⁽²⁾	Color	0.5 m
Infra-red images	2000 ⁽³⁾	Infra-red	0.35 m
	2000 ⁽⁴⁾	False-color	0.35 m

Notes: (1) ortho-rectified by GEO
 (2) ortho-rectified by Lands Department
 (3) compiled by Chinese University of Hong Kong
 (4) converted by GEO

GEOGRAPHIC INFORMATION SYSTEM (GIS)

Development of Capability

Geographic Information System (GIS) has been adopted in Hong Kong for several years in recording and managing geotechnical data. By nature, most geotechnical data contain spatial attributes on their geographic location (x, y, and z) and on the geometry of the ground/object (e.g. point, line or polygon) represented by the data. Managing the data in GIS would register the spatial attributes and permit the use of the attributes in GIS-related application, resulting in improved capability and efficiency.

Given its geotechnical mandate, since early 1990s the GEO has been a key party in developing geotechnical spatial datasets and GIS capability and promoting GIS application to geotechnical work in Hong Kong. In the early years, the development and application work focused on the following areas:

- (1) Compilation of GIS datasets, including conversion of existing data into GIS format and collation of new data. Important GIS datasets that are frequently used in geotechnical work (Figure 7) include:

- Topographic and land information maps covering the whole of HKSAR in various scales, up to 1:1,000 scale
- 1:20,000 scale geological maps covering the whole of HKSAR and 1:5,000 scale for some development areas
- A catalogue of 57,000 registered man-made slopes including slope related information
- An inventory of over 30,000 historical natural terrain landslides
- An inventory of about 2,000 large (scar > 20m wide) natural terrain landslide features
- An inventory of landslide incidents reported to the GEO since 1984
- 5-minute rainfall data since 1985 from about 100 nos. of raingauges
- Ground investigation data
- 5 m-grid DTM, which is being upgraded to 2 m-grid
- About 50 sets of ortho-rectified images covering the whole of HKSAR (Table 1)



Figure 7 – Selected geotechnical GIS datasets (Note: ortho-rectified infra-red image of year 2000, contour lines, registered man-made slopes, historical natural terrain landslides, reported landslide incidents and ground investigation data as shown)

- (2) Development of GIS systems and capability, such as:
 - Setting up software and hardware systems
 - Developing GIS skills among professional and technical users, typically through short courses and on-the-job training
 - Integrating GIS datasets and setting up centralized database for enhanced system management

- Acquiring Intranet, Internet and mobile GIS capabilities for ‘enterprise-based’ and other functional GIS applications, which involve use of specialized GIS modules, e.g. Spatial Database Engine and Internet Map Server for different scalable GIS applications. This development was however confined to key GIS users, such as the GEO and the Jockey Club Research and Information Centre (JCRIC) for Landslip Prevention and Land Development in the University of Hong Kong.
- (3) GIS applications, which mainly focused on data management and information services. A number of state-of-the-art GIS systems were set up in Hong Kong in the late 1990s and early 2000s, e.g. GEO’s Slope Information System (Figure 8), Fill Management System, Geological Modeling System and Aerial Photograph Management System (Figure 9), and the Comprehensive Ground Information System and Electronic Mark Plant Query System of JCRIC. These GIS systems are primarily used for data management and information services, with basic GIS search and browsing functionalities provided to users, without using a GIS software to interact with the systems.

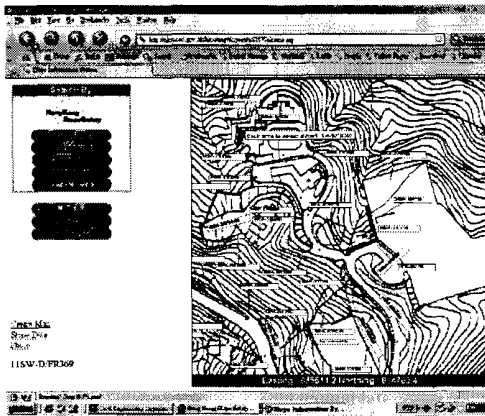


Figure 8 – Slope Information System

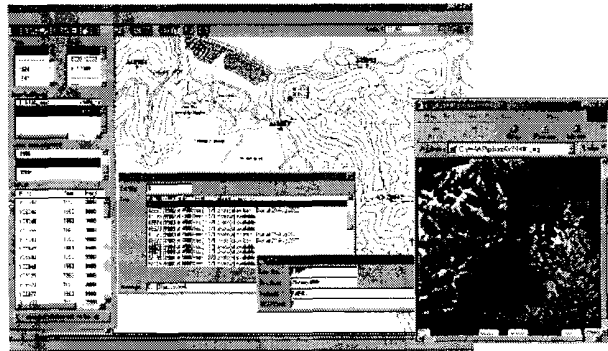


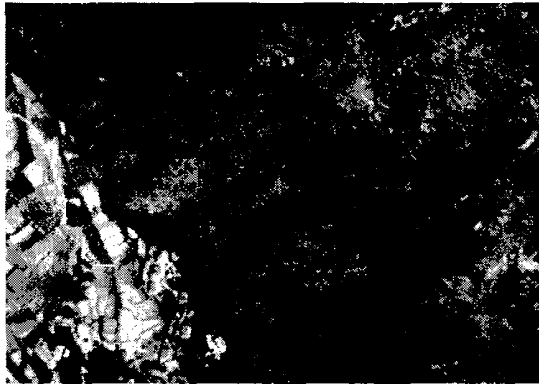
Figure 9 – Aerial Photograph Management System (Note: the system supports spatial search of aerial photographs covering the site concerned from over 120,000 nos. aerial photographs held in GEO’s Aerial Photograph Library)

Application Trend

While GIS data management and information services remain important, the trend in recent years is to adopt more advanced GIS functionality to deal with geotechnical issues in Hong Kong. This is possibly the result of building up of GIS capability among the geotechnical profession as well as increased demand for use of GIS in solving geotechnical problems. Some notable applications are described below:

- (1) Advanced GIS search, browsing, editing and publication
- This is performed by more skilled GIS users via the use of GIS tools, say, in a geotechnical desk study to examine the available geotechnical data, review the site history and assimilate the key information for presentation (Figure 10). Advanced GIS search would enable users to query and retrieve data that meet certain prescribed criteria or geographic relationship. Examples of such application include delineation of area of deep rock weathering using ground investigation data, search of man-made slope features for stability assessment, and identification of sites

affected by historical natural terrain landslides.



(a) Site in 1963

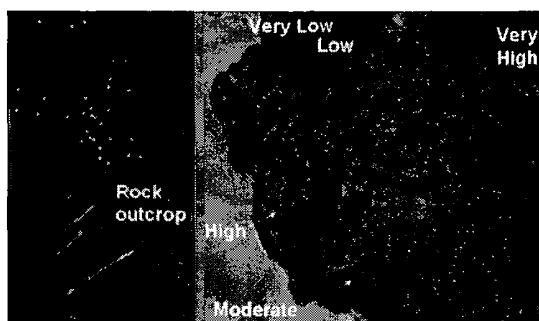


(b) Site in 2000 (Note: there are recent landslides on the natural hillside and new developments below the hillside)

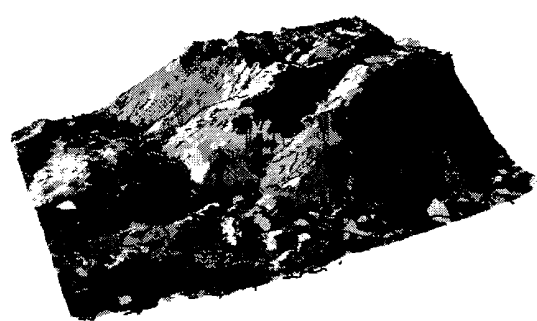
Figure 10 – Use of GIS in geotechnical desk study

(2) GIS analysis

GIS analysis can be performed efficiently to examine the relationship and correlation among different spatial data, which are difficult to analyze using conventional means. It is now used commonly in natural terrain landslide susceptibility analysis (Figure 11), as part of Natural Terrain Hazard Study (e.g. Evans and King 1998; Dai and Lee 2002; OPA 2003; Halcrow 2003; Wong 2004). GIS analysis also offers a unique capability in geotechnical research and development work involving spatial analysis of geotechnical data. Figure 12 shows a recent example where the correlation between natural terrain landslide density and rainfall intensity in Hong Kong has been established using GIS analysis together with GIS-based geo-statistics (Ko 2003).

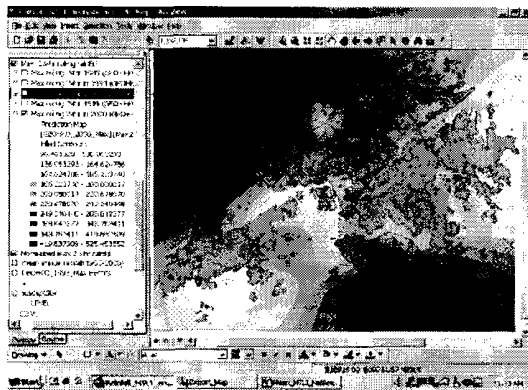


(a) GIS analysis and correlation of terrain with different levels of landslide susceptibility

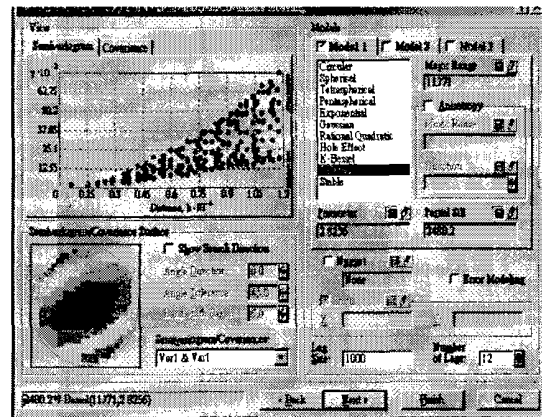


(b) 3-D visualization of historical landslides and terrain with different slope gradients

Figure 11 – GIS-based natural terrain landslide susceptibility analysis



(a) Year 2000 maximum rolling 24-hr rainfall and natural terrain landslide locations

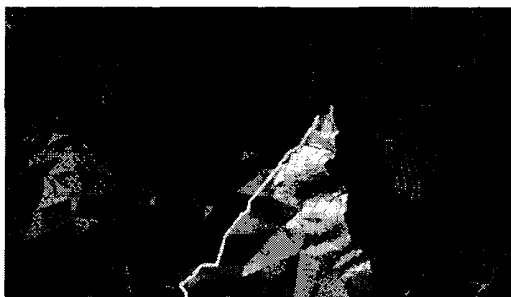


(b) GIS-based geo-statistical analysis

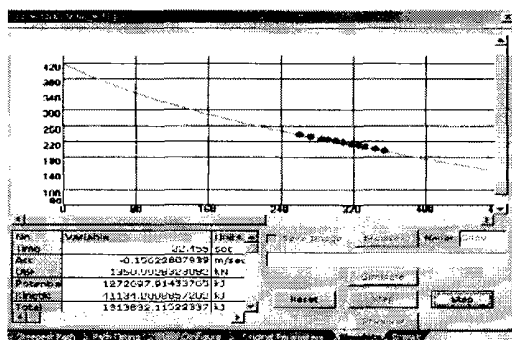
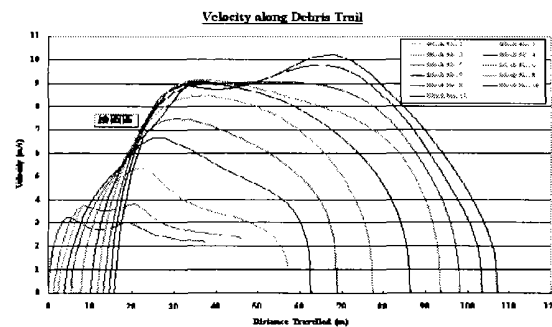
Figure 12 – GIS analysis of natural terrain landslide-rainfall correlation

(3) GIS modeling

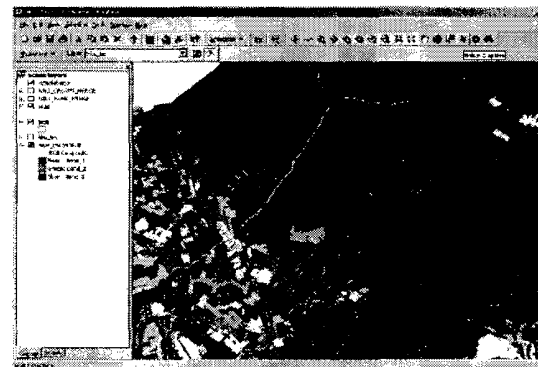
In addition to GIS analysis that is principally applied to assessing the spatial relationship among the data, performing GIS-based geotechnical analysis and numerical modeling based on application of engineering principles and governing physical laws has become increasingly important. Such application integrates engineering analysis with GIS, and is a powerful modeling tool, particularly for dealing with geotechnical subjects that involve the analysis of the geographic and engineering attributes of a large amount of spatial data. Development of GIS modeling applications requires GIS programming input from skilled personnel. Two geotechnical examples of such application in modeling the mobility of landslide debris and natural terrain landslide quantitative risk assessment are shown in Figures 13 and 14, respectively.



(a) Generation of debris flow path



(a) Generation of debris flow path



(c) Presentation of results

Figure 13 – GIS modeling of debris mobility

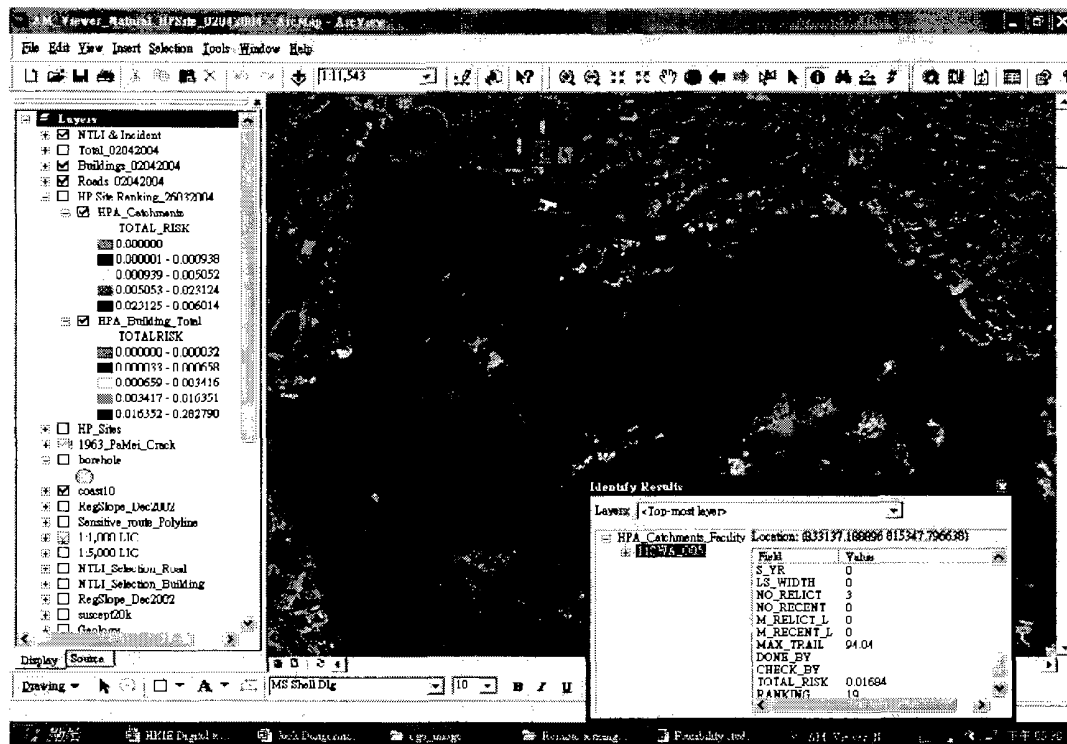


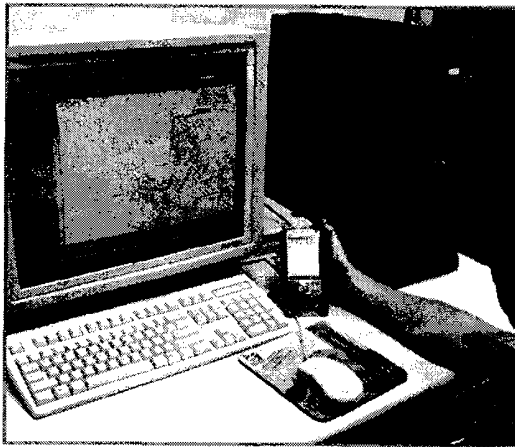
Figure 14 - GIS-based landslide quantitative risk assessment (Note: hazard and consequence models are incorporated into the GIS for calculation of risk using the relevant spatial data of the catchments and facilities at risk; catchments and facilities with different levels of calculated risk shown in different colours)

(4) Mobile, location-based application

Geotechnical professionals spend considerable time and effort in field work, which is an important component of geotechnical practice. GIS can now be brought to site and applied to field work, by uploading the relevant datasets onto a mobile GIS platform that operates on a pocket computer. When integrated with a Global Positioning System (GPS) for detecting the spatial location on site, a mobile GIS system can guide on-site navigation to the point of interest (Wong 2001). In addition, the spatial data relevant to the site can be retrieved for location-based applications. The GEO has pioneered the development of a state-of-the-art GIS-GPS mobile mapping system that also incorporates the use of ortho-imagery. The system is equipped with wireless telecommunication via the Internet with GEO's GIS Internet Map Server, for GIS data transfer for use in geotechnical field work (Figure15). This system was granted an international 'Special Achievement in GIS Award' in 2002 in recognition of its technological advances and benefits to the engineering field. Details of the system and its application are described in the paper by Ng et al (2004).

(5) 3-D visualization and virtual reality applications

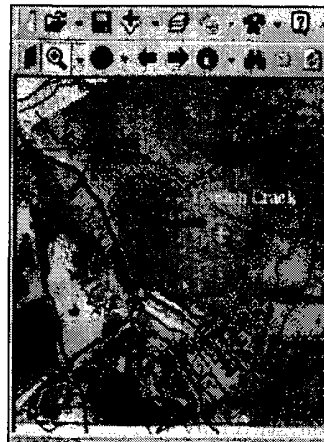
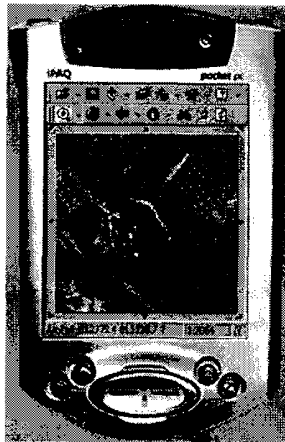
3-D GIS technology is maturing. Production of 3-D topography, city model, etc. has become relatively easy using the available spatial data, DTM and ortho-rectified images. Geotechnical professionals with GIS skills can now generate 3-D ground model and



(a) Uploading GIS data to the mobile mapping system



(b) On-site navigation, data retrieval and mapping using the mobile mapping system



(c) GIS data and ortho-rectified image together with GPS location shown on pocket computer

Figure 15 – Geotechnical mobile GIS mapping system

visualize the site conditions (Figure 16) on their desktop computers. This facilitates geotechnical studies, e.g. in reviewing historical landslides and terrain evaluation (Wong 2001). Virtual reality animation and computer 'fly-through' can be produced for presentation and evaluation purposes without the need for input from specialists in 3-D computer graphics.

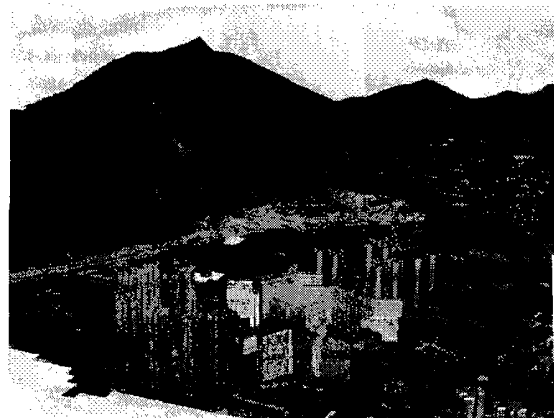


Figure 16 – 3-D virtual reality model of Tsing Shan Footfills

INTERFEROMETRIC SYNTHETIC APERTURE RADAR (InSAR)

Interferometric Synthetic Aperture Radar (InSAR) is an emerging remote sensing technology that could measure ground displacements with millimeter-level accuracy. Each pixel of a Synthetic Aperture Radar (SAR) image contains information on the phase of the signal backscattered from the terrain surface. By applying interferometry to a pair of SAR images of an area, the geometry of the two slightly displaced, coherent observations of the surface would give their phase difference, which is a function of the surface height. Through repeated observations, it is possible to measure the surface displacement if ground movement has occurred over the observation period.

InSAR has been successfully applied to measurement of surface movement over large area induced by earthquakes (e.g. Massonnet et al. 1993), volcanic activities (e.g. Lu et al. 2002), ground subsidence (e.g. Fielding et al. 1998), etc. Some cases of application to detection of slope movement have also been reported (e.g. Strozzi et al. 2001). It is potentially a promising low cost, high accuracy remote sensing technique for geotechnical use, particularly for ground and slope movement detection.

InSAR has been tried with encouraging results in Hong Kong in detection of reclamation settlement (Ding et al. 2004b). In collaboration with the Hong Kong Polytechnic University, the GEO has recently completed a trial application of InSAR for detection of slope movement and ground deformation at four selected sites in Hong Kong. The trial study shows that the available satellite-based SAR images are not very suitable for reliable InSAR analysis for the typical urban setting in Hong Kong (Figure 17). Further information on InSAR application in Hong Kong is reported in Ding et al. (2004a).

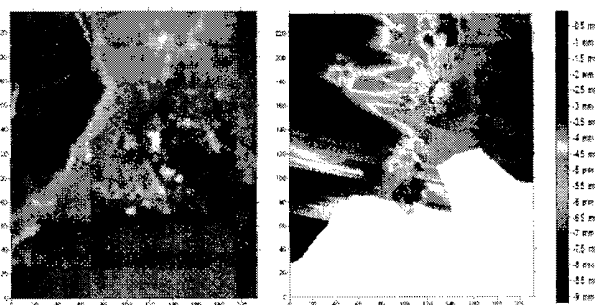


Figure 17 – Trial InSAR application for detection of slope movement in Hong Kong (18.3.1996 to 9.11.1999 3.6-year InSAR results at Tsing Shan Foothills, which are significantly affected by ‘noises’, as shown)

A realistic appreciation of the current limitations of the technique and good awareness of its development potential are essential to successful application. In this respect, the following constraints on using of satellite-based InSAR in Hong Kong are note-worthy:

- Many of the available SAR images have large Doppler Centroid Frequency and low coherence, particularly for long time-span. These result in poor quality interferograms, which affect the reliability of InSAR analysis.
- SAR images have low spatial resolution (each pixel typically > 10 m). Hence, the interferogram only shows the average condition for each pixel and any surface movement that may be measured is at best representing the overall movement in the pixel. Hence, InSAR is more suitable for use in spatially extensive sites that are without drastic changes in surface profile and significant variations in ground movement over short distance. If the site to be measured is small in size (e.g. a relatively small-sized man-made slope) or if local changes in ground profile and ground movement are significant, it is difficult to apply satellite-based InSAR.

- InSAR results can be seriously affected by noises, such as those arising from geometric distortion, atmospheric effects and temporal decorrelation. Geometric distortion can be a serious problem for applying InSAR to steep terrain in Hong Kong. Also, the relatively humid environment and presence of thick vegetation imply that atmospheric effects and temporal decorrelation would be more significant in Hong Kong, than in other countries that have a more favorable site setting for successful use of InSAR.

The quality and availability of SAR images would improve as more satellites are launched. Development of airborne and land-based InSAR would reduce noise effects and enhance the accuracy and spatial resolution of InSAR results. InSAR technologies, e.g. use of reflectors, filters and permanent scatter techniques, are evolving. The GEO is arranging the installation of a number of corner reflectors in selected sites in Hong Kong. This will provide an opportunity for testing the improvement that may be achieved and for developing the long-term potential for geotechnical application of InSAR in Hong Kong.

LIGHT DETECTION AND RANGING (LIDAR)

Light Detection and Ranging (LIDAR) is a promising remote-sensing tool for scanning surface topography by measuring the direction and time of sending and receiving coherent laser beams to the objects of interest. State-of-the-art land-based LIDAR capability is available in Hong Kong. Such land-based LIDAR system, commonly denoted as ‘laser scanner’ in Hong Kong, has the capability of measuring, 3-dimensional point clouds of objects within about 150 m along the line-of-sight. The laser scanner emits thousands of laser beam pulses per second for measuring a ‘window’ of 3-dimensional surfaces. The positional accuracy is within 6 mm in a 50-m range. The point clouds, apart from providing spatial information on their x, y, and z coordinates, contain an intensity signal of the laser reflection and hence present a 3-D false-color digital model of the scanned object.

The GEO has been using the laser scanner in topographic surveys, which is particularly useful where physical access to the survey site is difficult or dangerous, e.g. a new landslide scar (Figure 18). Similar application of laser scanner in surveying a full-scale test of fill slope failure has also been reported by Kwong (2003).

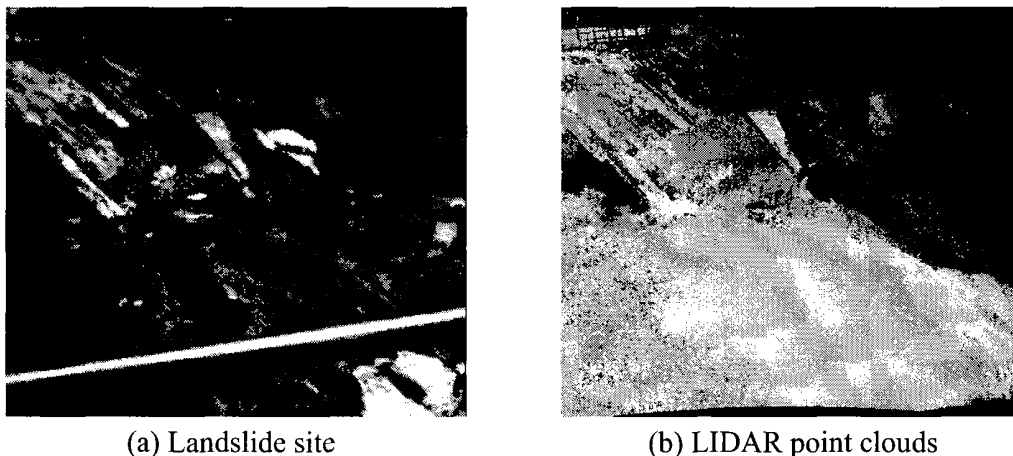
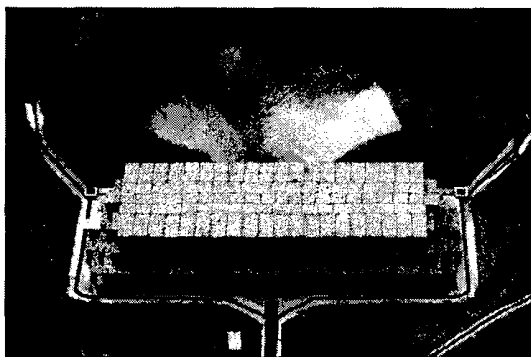


Figure 18 – LIDAR survey of a landslide site

LIDAR can be used in many novel geotechnical applications apart from topographic survey, such as:

- (1) Construction of high-resolution DTM – given the high sampling density, DTM produced by LIDAR can enhance the quality and supplement the DTM produced from the available topographic maps and from digital photogrammetry.
- (2) Compilation of 3-D digital models of slopes, debris-resisting barriers, structures and other geotechnical features (Figure 19) – this assists construction monitoring and provides an accurate and detailed virtual reality records for use in future maintenance and modification work.



(a) Debris-resisting barrier



(b) 3-D digital LIDAR model

Figure 19 – 3-D digital model compiled by LIDAR

- (3) Movement monitoring of slopes and structures (Figure 20) – movement can be detected and monitored by comparing LIDAR results obtained at different time; there is however a need to register the key data of common observation points in different LIDAR surveys to ensure accuracy and efficiency.

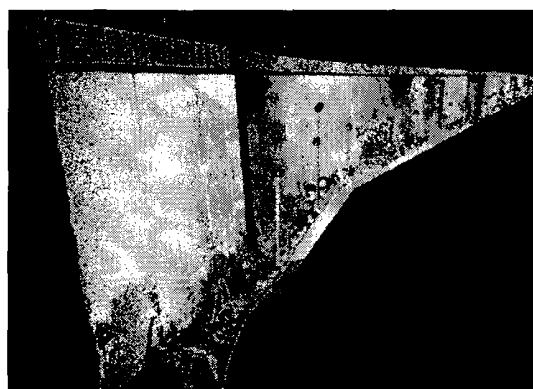
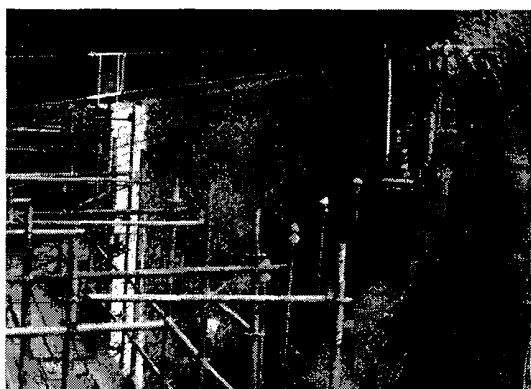


Figure 20 – Use of LIDAR in monitoring movement of a retaining wall

- (4) Rock slope mapping and rock joint survey (Figure 21) – by judiciously matching and analyzing the LIDAR point clouds.

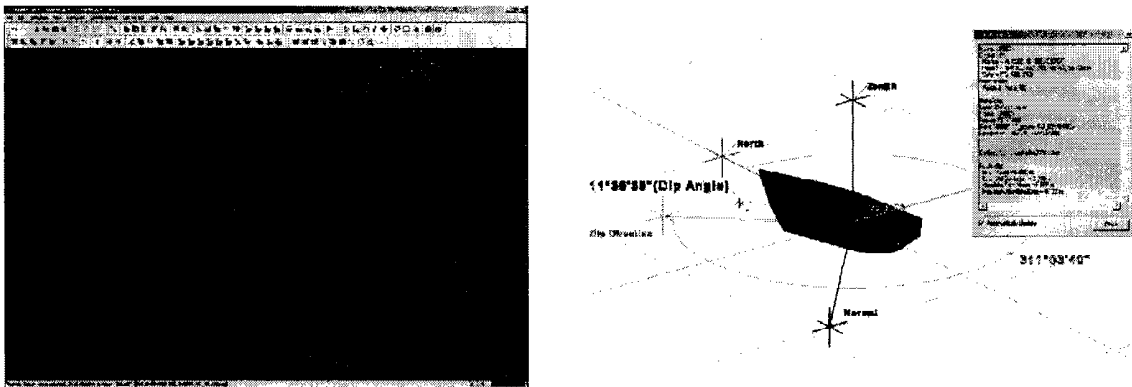


Figure 21 – Use of LIDAR in rock joint mapping and survey (Note: the blue area is a joint plane on a rock face with the joint orientation measured by LIDAR)

Customization and development work, which is essential for tailor-making the available technique for efficient and effective geotechnical use, is being pursued by the GEO and local academic institutions. Table 2 summarizes the key customization and development work currently undertaken by the GEO on the novel LIDAR applications identified (Wong 2003).

Table 2 – LIDAR Customization and Development Work (extracted from Wong 2003)

Customization and Development Work	Applicable to Item No.			
	(1)	(2)	(3)	(4)
Optimization of the techniques and processes for re-sampling and conversion of the LIDAR data, for use on other digital platforms, e.g. GIS, CAD and digital photogrammetry systems	✓	✓	✓	✓
Building of slope and structure monitoring models using LIDAR data, and development of methods for change detection		✓	✓	
Development of methodology for LIDAR mapping of rock slopes and survey of rock joints, possibly in combination with use of digital photogrammetry and image analysis techniques				✓
Assessment of the optimal details to be contained in 3-D virtual reality records, and examination of suitable methods and platforms for use in compiling and managing the records		✓	✓	

Legend: (1) = Construction of DTM
 (2) = Compilation of 3-D digital models
 (3) = Movement monitoring
 (4) = Rock slope mapping and joint survey

Airborne LIDAR, which can survey a large area efficiently and at competitive cost, is subject to active technological development in other advanced countries where there is a demand for high-resolution topographic data. Airborne LIDAR is typically performed by

mounting a high-powered LIDAR at the bottom or sides of a plane/helicopter to scan the ground features along the flight path. The instrument is bundled with accurate onboard differential GPS (DGPS) to register the reference plane and positioning during the LIDAR survey. It has been used in regional topographic mapping and construction of DTM, typically with an accuracy of ± 0.15 m in height value. An important recent development is the use of a multi-return LIDAR to measure multiple returns for each laser pulse that covers several feet in diameter on ground. With the use of an advanced numerical algorithm, the last returns that come from the ground surface are extracted by filtering other returns from vegetation and building structures (a technique known as 'virtual deforestation'). Hence, the system has the capability of mapping the ground surface of vegetated terrain (Haugerud and Harding 2003). The technique has been used to produce fine-scale topographic maps and DTM typically with grid size of about 1 m. This allows landslide geomorphology to be interpreted and landslide maps to be produced to a resolution that cannot otherwise be achieved by using conventional aerial photographs. It is recognized as one of the most important remote-sensing tool for obtaining high-quality digital elevation data and for landslide hazard mapping (National Research Council 2004). However, its applicability in Hong Kong is yet to be tested.

CONCLUSIONS

Hong Kong has been actively pursuing the development and geotechnical application of novel digital technologies. This has led to improved capability and efficiency. The geotechnical profession should continue to maintain awareness of the technological development and opportunities for geotechnical application. Digital photogrammetry and GIS are examples of novel digital technologies that have successfully been integrated into our geotechnical practice. InSAR and LIDAR are emerging techniques that have notable potential. Development and customization work, which will further enhance their capability and suitability for geotechnical application, is being undertaken in Hong Kong and elsewhere.

REFERENCES

- Dai, F.C. and Lee, C.F. (2002). "Terrain-based mapping of landslide susceptibility using a geographic information system: a case study." *Canadian Geotechnical Journal*, Vol. 38, 911-923.
- Ding, X.L., Li, Z.W., Liu, G.X., Yin, J.H. and Chen, Y.Q. (2004a). "Applying synthetic aperture radar interferometry technology to detection of ground movement." *Proceedings of the Seminar on Recent Advances in Geotechnical Engineering*, Geotechnical Division, Hong Kong Institution of Engineers, (in press).
- Ding, X.L., Liu, G.X., Li, Z.W., Li, Z.L. and Chen, Y.Q. (2004b). "Ground settlement monitoring in Hong Kong with satellite SAR interferometry." *Photogrammetric Engineering and Remote Sensing*, (in press).
- Evans, N.C. and King, J.P. (1998). "The Natural Terrain Landslide Study: Debris Avalanche Susceptibility." *Technical Note No. TN 1/98*, Geotechnical Engineering Office, Hong Kong, 96 p.
- Fielding, E.J., Blom, R.G., and Goldstein, R.M. (1998). "Rapid subsidence over oil fields measured by SAR interferometry." *Geophysical Research Letters*, 25(17), 3215-3218.

- Halcrow China Limited (2003). "Detailed Study of Selected Natural Terrain Landslides at Cloudy Hill." *Landslide Study Report No. LSR 6/2003*, Geotechnical Engineering Office, Hong Kong, 91 p.
- Hansen, A and Lichti, D.D. (2002). "Using digital imagery to obtain geomorphological information for landslide susceptibility assessment." *Proceedings of the Seminar on Natural Terrain – A Constraint to Development*. Institution of Mining and Metallurgy, Hong Kong Branch, Hong Kong, 101-112.
- Haugerud, R.A. and Harding, D.J. (2003). "Some algorithms for virtual deforestation (VDF) of LIDAR topographic survey data."
- Ko, F.W.Y. (2003). "Correlation between rainfall and natural terrain landslide occurrence in Hong Kong." Special Project Report No. SPR 7/2003. Geotechnical Engineering Office, Hong Kong, 74 p.
- Kwong, A.K.L. (2003). "Application of 3D laser scanning to slope movement monitoring." *Proceedings of the Third Cross-Strait Conference on Structural and Geotechnical Engineering*, 23 – 25 October 2003, Taiwan.
- Li, K.S. and Mo K.W. (2003). "Use of photogrammetry for monitoring of rubble structures." *Proceedings of the Seminar on Intelligent Engineering Applications of Digital Remote Sensing Technology*. Civil, Environmental and Geotechnical Divisions, Hong Kong Institution of Engineers, Hong Kong, 29-34.
- Lu, Z., Wicks, C., Dzurisin, D., Power, J.A., Moran, S.C., and Thatcher, W. (2002). "Magmatic inflation at a dormant stratovolcano: 1996-98 activity at Mount Peulik volcano, Alaska, revealed by satellite radar interferometry." *Journal of Geophysical Research*, 103(B12), 2134-2146.
- Massonnet, D., Rossi, M., Carmona, C., Adragna, F., Peltzer, G., Feigi, K. and Rabaute, T. (1993). "The displacement field of the Landers earthquake mapped by radar interferometry." *Nature*, 364, 138-142.
- National Research Council (2004). "Partnership for Reducing Landslide Risk – Assessment of the National Landslide Hazards Mitigation Strategy." National Academies Press, Washington, D.C., 131 p.
- Ng, K.C., Fung, K.S. and Shum, W.L. (2004). "Applying mobile GIS technology to geotechnical fieldwork". *Proceedings of the Seminar on Recent Advances in Geotechnical Engineering*, Geotechnical Division, Hong Kong Institution of Engineers, (in press).
- Ove Arup and Partners Hong Kong Limited (2003). Natural Terrain Hazard Study at Pat Heung, Yuen Long. Advisory Report No. 1/2003, Geotechnical Engineering Office, Hong Kong, 266 p.
- Shi, W.Z., Ng, K.C., Huang, C.Q. and Li, X.C. (2004). "Applying image processing technology to boulder mapping." *Proceedings of the Seminar on Recent Advances in Geotechnical Engineering*, Geotechnical Division, Hong Kong Institution of Engineers, (in press).
- Slope Safety Technical Report Board (2003). Slope Safety Technical Review Board No. 12. Geotechnical Engineering Office, Hong Kong, 40 p.
- Slope Safety Technical Report Board (2004). Slope Safety Technical Review Board No. 13. Geotechnical Engineering Office, Hong Kong, 41 p.
- Strozzi, T., Wegmuller, U., Tosi, L., Bitelli, G. And Spreckels, V. (2001). "Land subsidence monitoring with differential SAR interferometry." *Photogrammetric Engineering & Remote Sensing*, 67(11), 1261-1270.

- Wong, H.N. (2001). Recent advances in slope engineering in Hong Kong. (Invited paper). Proceedings of the 14th Southeast Asian Geotechnical Conference, Hong Kong, Vol. 1, 641-659.
- Wong, H.N. (2003). "Briefing paper to Slope Safety Technical Review Board – Natural Terrain Landslide Risk Management and Application of Digital Technology." Geotechnical Engineering Office, 34 p, (unpublished).
- Wong, H.N. (2004). Natural terrain management criteria – Hong Kong practice and experience. (Invited paper). Proceedings of the International Conference on Fast Slope Movements: Prediction and Prevention for Risk Mitigation, Naples, Italy, Vol. 2, (in press).

ACKNOWLEDGEMENTS

The paper is published with the permission of the Head of the GEO and the Director of Civil Engineering, Government of the HKSAR. Contribution from members of the Interest Group on Application of Innovative Technology in Geotechnical Engineering and other colleagues of the GEO, who have taken part in pursuing the application of the technologies described in this paper, are gratefully acknowledged.

APPLYING SYNTHETIC APERTURE RADAR INTERFEROMETRY TECHNOLOGY TO DETECTION OF GROUND MOVEMENT

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Abstract: Satellite Interferometric Synthetic Aperture Radar (InSAR) is a space-based earth observing technology that has undergone major development in the past decade. The technology has a number of advantages, especially in its large ground coverage, competitive measurement accuracy and reasonable spatial resolution. This paper reports the results from some experimental studies carried out in Hong Kong to detect ground movement with InSAR. The studies show that successful use of the technology is highly dependent on the atmospheric conditions and on the ground features including the type of surface coverage and the surface geometry. Under good conditions, it is possible to accurately measure ground movement over a period of several years. However, it could be difficult to get reliable results over a few months if the conditions are less favorable.

PRINCIPLES OF SAR INTERFEROMETRY FOR GROUND MOVEMENT MONITORING

Synthetic aperture radar interferometry (InSAR) has recently been demonstrated to be a promising technology for mapping ground deformations without requiring laborious field accesses (e.g., Massonnet and Feigl 1998). It uses multiple SAR images acquired at different times over the same region to infer ground displacements over the region. InSAR is an area-based measurement technique that offers much higher spatial resolution (decimetres), wide spatial coverage (thousands of square kilometres with one image scene), and very competitive accuracy (millimetres to centimetres) under good conditions. In addition, InSAR works day and night and under all-weather conditions (although affected by the weather conditions to certain extent), and is highly automatic in data processing. The special characteristics of InSAR make it a convenient, efficient and economical tool to study many geophysical and geotechnical problems. InSAR has been successfully applied to measure ground motion caused by, e.g., earthquakes, landslides, land settlement, underground water pumping, underground mining, oil extraction, subway construction, thermal field exploitation (Massonnet et al. 1993; Fruneau et al. 1996; Massonnet et al. 1997; Fielding et al. 1998; Carnec and Delacourt 2000; Tesauro et al. 2000; Liu et al. 2001; Strozzi et al. 2001; Ding et al. 2004; Liu et al. 2004).

SAR Interferometry for ground movement monitoring begins with the generation of interferograms by complex conjugate multiplication of two co-registered SAR images:

$$\text{INTF}(r, a) = M(r, a) \cdot S(r, a)^* \quad (1)$$

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where (r, a) are image coordinates in range and azimuth, respectively; M and S are the complex master and the re-sampled slave image, respectively; $*$ denotes the complex conjugate operation. The phases of the generated interferogram (or phase differences between the master and slave images) are the principal observables in InSAR measurements. Assuming that $A1$ and $A2$ are the positions of radar platform corresponding to two acquisitions (see Figure 1), the interferometric phase ϕ is then

$$\phi = \psi_1 - \psi_2 = \frac{4\pi}{\lambda} L_1 - \frac{4\pi}{\lambda} L_2 \quad (2)$$

where L_1 and L_2 are the slant ranges and λ is the wavelength of the signal.

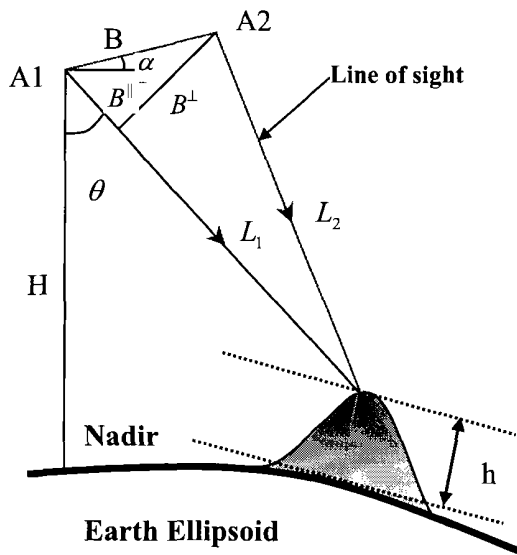


Figure 1: Interferometric geometry

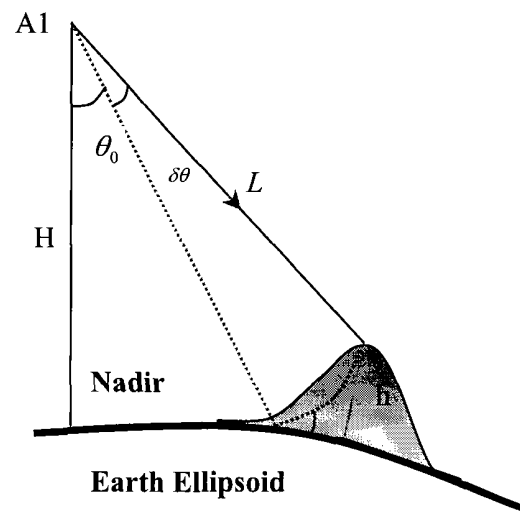


Figure 2: Phase flatten geometry

Under the far field approximation, one gets

$$\phi = \psi_1 - \psi_2 \approx \frac{4\pi}{\lambda} B \sin(\theta - \alpha) \quad (3)$$

where α is the orientation angle of the baseline and θ is the look angle.

When assuming a surface without topographic relief as illustrated in Figure 2, the interferometric phase becomes

$$\phi_0 = \frac{4\pi}{\lambda} B \sin(\theta_0 - \alpha) \quad (4)$$

where θ_0 is the look angle. If topographic relief is present, the look angle will differ from θ_0 by $\delta\theta$:

$$\phi = \frac{4\pi}{\lambda} B \sin(\theta_0 + \delta\theta - \alpha) \quad (5)$$

Combining Equations (4) and (5), we get the "flattened" phase

$$\phi_{flat} = \phi - \phi_0 \approx \frac{4\pi}{\lambda} B \cos(\theta_0 - \alpha) \delta\theta = \frac{4\pi}{\lambda} B^\perp \delta\theta \quad (6)$$

The relationship between the topographic height and $\delta\theta$ can be easily established (see Figure 2)

$$h \approx L \delta\theta_0 \cdot \sin \theta_0 \quad (7)$$

Thus the topographic height can be expressed as

$$h = \frac{\lambda L}{4\pi B \cos(\theta_0 - \alpha)} L \phi_{flat} \quad (8)$$

When ϕ_{flat} reaches one cycle(2π rad.), the corresponding topographic height is the ambiguity height and can be expressed as:

$$h_{2\pi} = \frac{\lambda L}{2B \cos(\theta_0 - \alpha)} \sin \theta_0 \quad (9)$$

The aforementioned process is based on the assumption that the imaged surface is stationary during the acquisitions. Note that the phase difference in repeat-pass interferometry includes ground displacement in addition to topography. Differential InSAR or differential SAR interferometry (D-InSAR) is the technique to extract displacement signature in SAR interferogram over the acquisition period.

In Figure 3, an exaggerated ground displacement Δd between two acquisitions is shown. The projection of the displacement onto the radar line-of-sight direction is Δr . The displacement introduces a variation of interferometric phase that is proportional to Δr and can be expressed as follows:

$$\Delta\phi_{flat} = \frac{4\pi}{\lambda} \Delta r \quad (10)$$

Therefore, the phase difference that included topography information as well as deformation information becomes

$$\phi_{flat} \approx \frac{4\pi}{\lambda} \frac{B \cos(\theta_0 - \alpha)}{L \sin \theta_0} h + \frac{4\pi}{\lambda} \Delta r \quad (11)$$

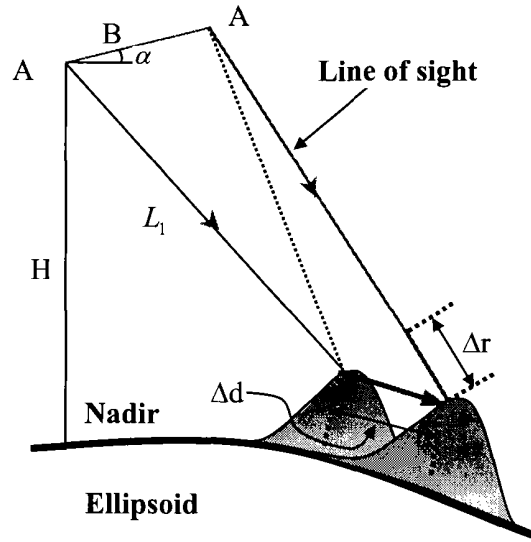


Figure 3: Geometry of differential SAR interferometry

To map the deformation between the acquisitions, the topographic contribution must be removed. According to the ways for removing the topographic contribution, three types of D-InSAR configurations can be distinguished, i.e., (1) two-pass plus an external DEM, (2) three-pass, and (3) four-pass.

In the two-pass plus an external DEM approach, a SAR interferogram (“topographic” interferogram) is simulated based on the DEM and the imaging geometry of the ‘real’ interferogram (“deformation” interferogram) and thus removed from the deformation interferogram. However, in the three-pass and four-pass formulations, both the “topographic” and the “deformation” interferograms are generated from the SAR images. The difference between the two approaches is that in the three-pass interferometry, one image is shared by both the “topographic” and the “deformation” interferogram. The formulae used for D-InSAR for the two-pass plus an external DEM and for the three-pass and four-pass configurations are:

$$\Delta r_{two} = \frac{\lambda}{4\pi} (\phi_d - \phi_{sim,t}) \quad (12)$$

$$\Delta r_{three,four} = \frac{\lambda}{4\pi} (\phi_d - \frac{B_d^{\parallel}}{B_t^{\parallel}} \phi_t) \quad (13)$$

where ϕ_d and ϕ_t are phases of the deformation and the topography interferograms, respectively, and $B_d^{\parallel}$ and $B_t^{\parallel}$ are the line-of-sight baseline component of the deformation and topography interferograms, respectively.

The interferometric quality of an interferogram can be measured to a large extent by the associated coherence map, i.e., signal correlation coefficients $\hat{\gamma}$ that can be calculated using a number of neighboring pixels within a N-size window through the following estimator,

$$\hat{\gamma} = \left| \frac{\sum_{i=1}^N M_i \cdot S_i^*}{\sqrt{\sum_{i=1}^N M_i \cdot M_i^* \sum_{i=1}^N S_i \cdot S_i^*}} \right|. \quad (14)$$

The temporal decorrelation and atmospheric effects are two main uncontrollable limiting factors that degrade the interferometric quality.

It is important to realise that the phase value at each pixel in a differential interferogram is ambiguous to within the integer multiples of 2π , and that the phase in an absolute sense needs determined by the so-called phase unwrapping algorithms. The derived absolute phase values ($\Delta\phi_{abs}$) is converted to the LOS range change (Δr) by,

$$\Delta r = \frac{\lambda}{4\pi} \cdot \Delta\phi_{abs} \quad (15)$$

In the absence of horizontal movement, the vertical movement Δh can be calculated with

$$\Delta h = \frac{\Delta r}{\cos \theta} = \frac{\lambda \cdot \Delta\phi_{abs}}{4\pi \cdot \cos \theta} \quad (16)$$

In the case of ERS-1/2 C band ($\lambda=5.6\text{cm}$) SAR system (nominally $\theta = 23^\circ$), it is readily derived with equation (16) that the sensitivity of the phase with respect to ground subsidence is about 3.0cm, i.e., one full phase cycle (2π) can detect 3.0cm vertical change. As for JERS-1 L band ($\lambda=23.5\text{cm}$) SAR system (nominally $\theta = 35^\circ$), however, the corresponding sensitivity becomes 14.3cm. Given a phase RMS error of 40° , the subsidence RMS error is about 3.0mm for the ERS-1/2 images, and 15.9mm for the JERS-1 images. It is clear from the analysis that the longer the radar wavelength is, the less sensitive the system is to ground subsidence. On the other hand, long wavelength data are more robust in terms of maintaining temporal correlation over vegetated areas.

Past analysis has shown that the suitability of an interferometric pair for observing the Earth mainly depends on their orbital separation, or more specifically, on the length of the baseline component perpendicular to the radar LOS (sometimes termed as effective baseline). For the purpose of ground movement monitoring, the shorter the baseline, the better. This on one hand can reduce the baseline decorrelation effect. On the other hand, in the case of two-pass plus an external DEM approach, short baseline can mitigate the effects of errors in the DEM on the final differential results. For example, an error (δ_{DEM}) in the DEM will cause a phase error of

$$\delta\phi = \frac{4\pi}{\lambda} \cdot \frac{B_{\perp}}{L \cdot \sin \theta} \cdot \delta_{DEM} \quad (17)$$

in the two-pass plus an external DEM approach. It is easy to see from the above equation that $\delta\phi$ approaches 0 when B does. Therefore a short baseline can limit the propagation of DEM errors.

The data processing procedures involved in differential SAR interferometry for ground movement monitoring is schematically displayed in Figure 4.

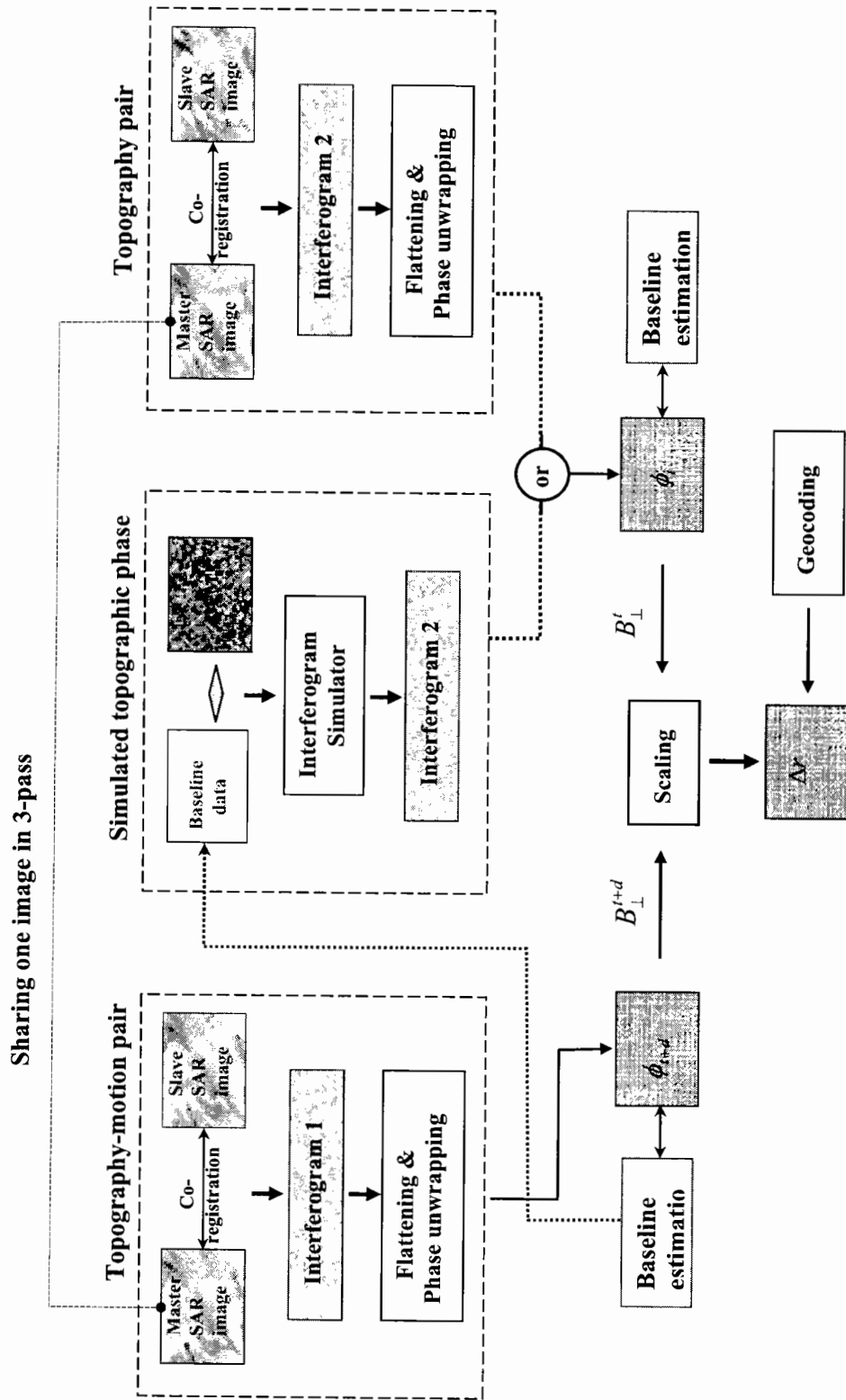


Figure 4: Data processing procedures of three D-InSAR approaches

SAR IMAGES USED FOR THE STUDY

The following ERS SAR images are used for this study:

Table 1: ERS SAR images available

No.	Sat.	Date	Doppler Centroid Frequency		
			a_0 (Hz)	a_1 (Hz/pixel)	a_2 (Hz/pixel ²)
1	ERS1	18/03/ 1996	129.67	0.0042350	-1.38257e-006
2	ERS2	19/03/1996	-194.638	0.0097792	-8.23008e-007
3	ERS2	02/07/1996	-105.739	-0.0017100	-8.25094e-007
4	ERS2	24/11/1998	-69.76	0.0055148	-3.01677e-007
5	ERS2	29/12/1998	-117.629	0.0117174	-1.40134e-006
6	ERS2	09/11/1999	-75.537	0.0143400	-1.71414e-006
7	ERS2	31/07/2001	1047.86	0.0632634	-3.0557e-006
8	ERS2	07/05/2002	3673.63	0.1505920	-2.73525e-006
9	ERS2	11/06/2002	3356.76	0.1542500	-4.13937e-006

Out of these images, images Nos. 7-9 have very large Doppler Centroid Frequency that prevented the formation of meaningful interferograms. 15 possible interferograms (Table 2) can be formed with the remaining six images. The perpendicular baselines of the interferograms and the time interval between the images are listed in the table.

COHERENCE ANALYSIS

The coherence images of some selected interferograms (those in Table 2 with their time intervals underlined) are given in Figures 5-10. From these images, one can see that in general, the coherence level decreases rapidly with the increase of the time interval between a pair of images and that only interferograms with very short time intervals, e.g., those with intervals of 1 and 35 days, maintain satisfactory coherences over most of the areas in Hong Kong.

Table 2: Interferometric combinations and their baselines

Slave Master	19/03/1996	02/07/1996	24/11/1998	29/12/1998	09/11/1999
18/03/1996	101* (1 day)**	-181 (106 days)	480 (2.7 years)	425 (2.8 years)	-55 (3.6 years)
19/03/1996		-282 (105 day)	379 (2.7 years)	324 (2.8 years)	-156 (3.6 years)
02/07/1996			661 (2.4 years)	606 (2.5 years)	116 (3.3 years)
24/11/1998				55 (35 days)	-535 (350 days)
29/12/1998					-480 (315 days)

* Perpendicular Baseline

** Time interval

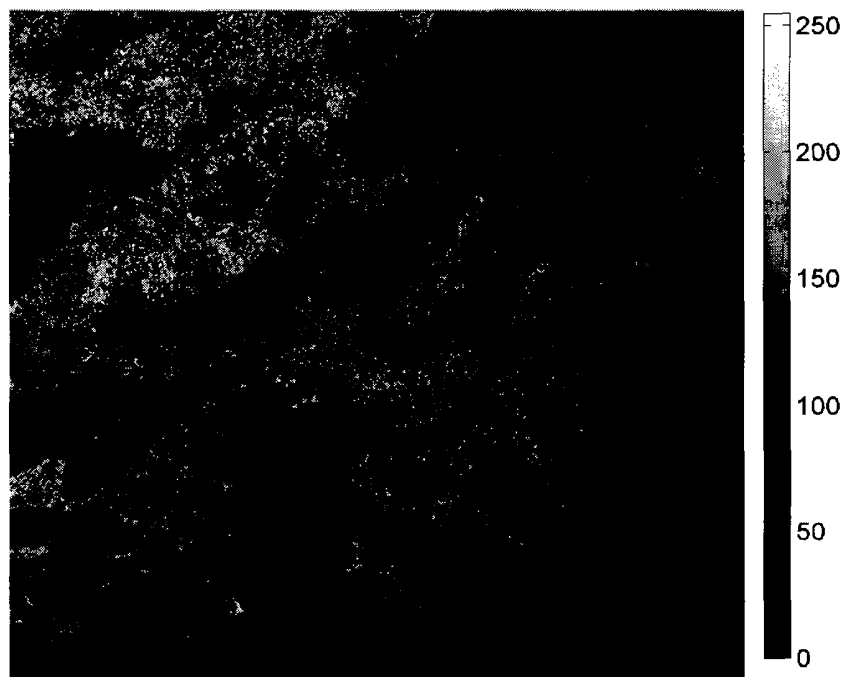


Figure 5: Coherence image of interferogram with a one-day interval (18/03/1996 - 19/03/1996)

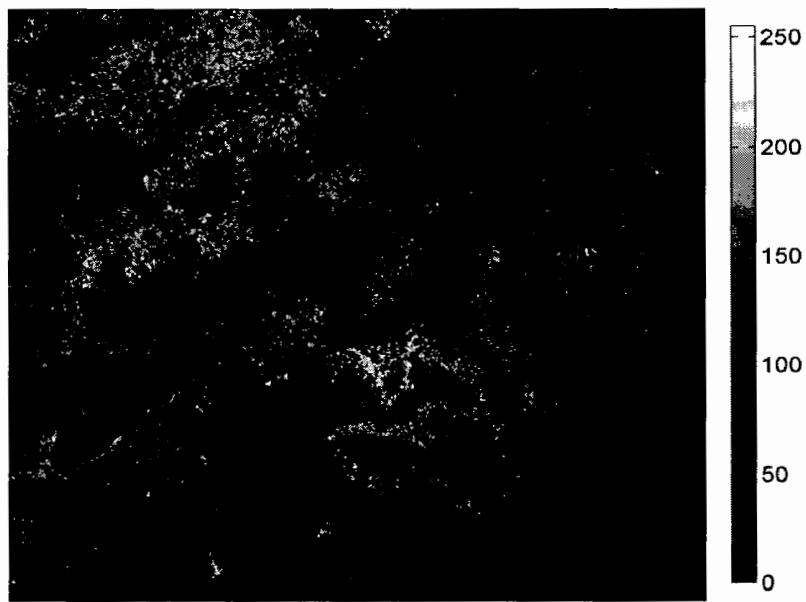


Figure 6: Coherence image of interferogram with a 35-day interval (24/11/1998 - 29/12/1998)

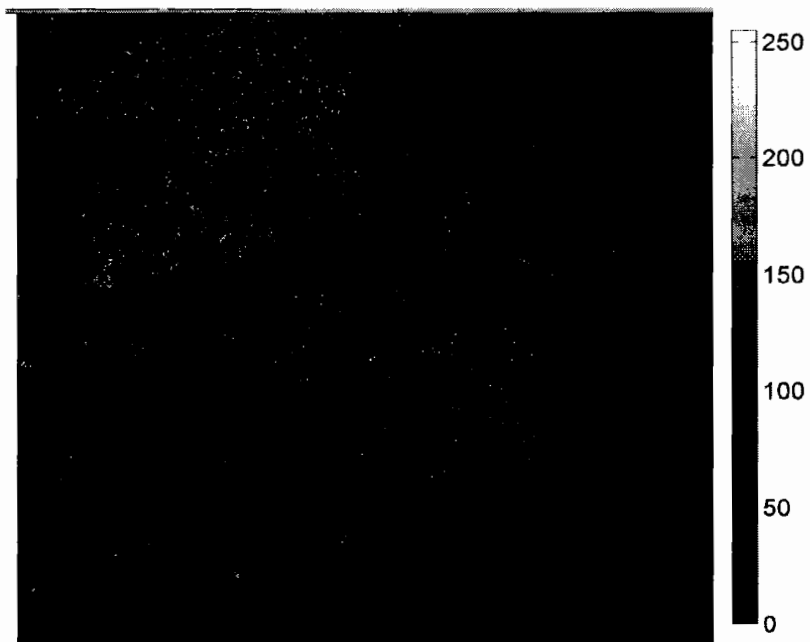


Figure 7: Coherence image of interferogram with a 106-day interval (18/03/1996 - 02/07/1996)

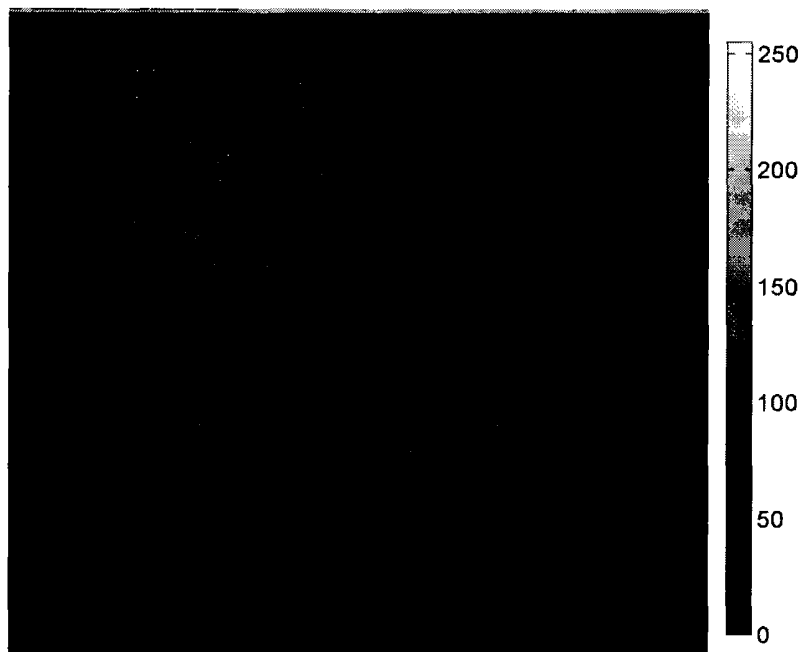


Figure 8: Coherence image of interferogram with a 315-day interval (29/12/1998 - 09/11/1999)

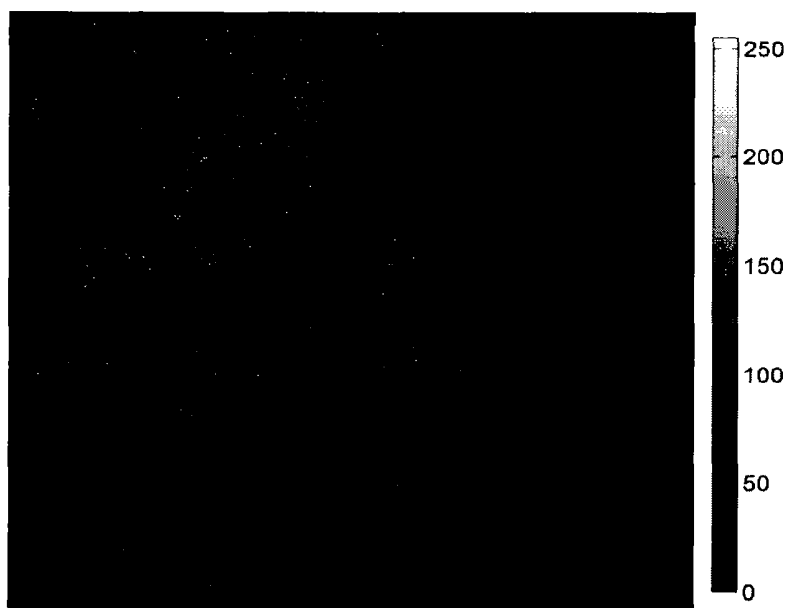


Figure 9: Coherence image of interferogram with a 2.8-year interval (19/03/1996 - 29/12/1998)

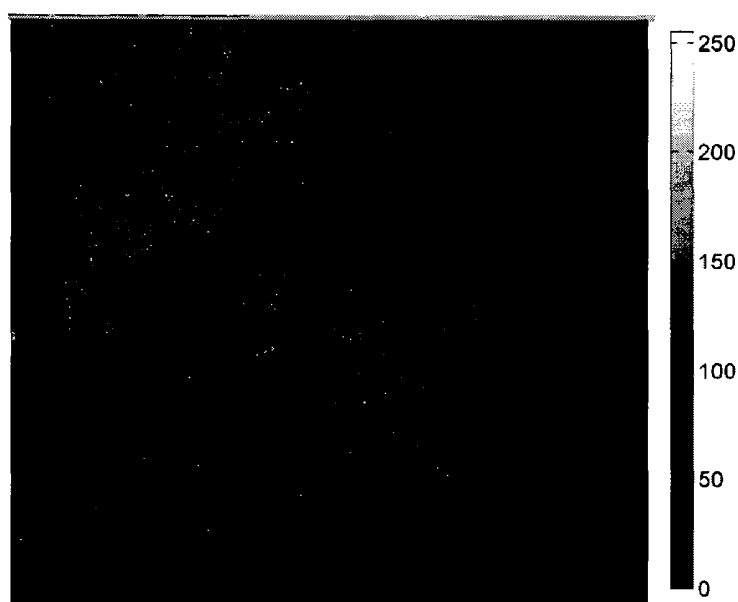


Figure 10: Coherence image of interferogram with a 3.6-year interval (18/03/1996 - 09/11/1999)

CASE STUDIES

Data Processing Method Adopted

The two-pass plus an external DEM method was adopted in processing the data. The processing steps are:

- (1) Co-registration of two SAR images;
- (2) Co-registration of external DEM to master SAR image;
- (3) Forming and filtering of differential interferogram;
- (4) Removal of flat-earth phase trend;
- (5) Phase unwrapping; and
- (6) Geocoding.

The DEM used is created from a digital map from the Lands Department, Government of the HKSAR. The RMS error of the DEM is smaller than 0.5m. The phase unwrapping is carried out with the branch-cut method (Goldstein et al. 1988).

Settlement in a Reclaimed Residential Area

Fairview Park (Figure 11) in Yuen Long is used as an example of settlement monitoring of reclaimed land. The area maintains good coherence (up to 0.65 on average) in all the interferometric combinations, even for the pair with an interval of nearly 3 years. This is considered mainly due to the special shapes and finishing of the roofs of the low-rise buildings in



Figure 11: A residential area, Fairview Park in Yuen Long

the area. Figure 12 shows a 3D-settlement map derived from the SAR image pair acquired on 19/03/1996 and 29/12/1998. The results obtained show that the area was settling between 1996 and 1999. It is very interesting to note that the subsidence (about 40mm) on the side near the sea is much larger than that (about 0mm) on the side farther from the sea, and that the gradual variations across the area are clear. The InSAR analysis results at this site are consistent although their reliability is yet to be validated by any available survey monitoring results.

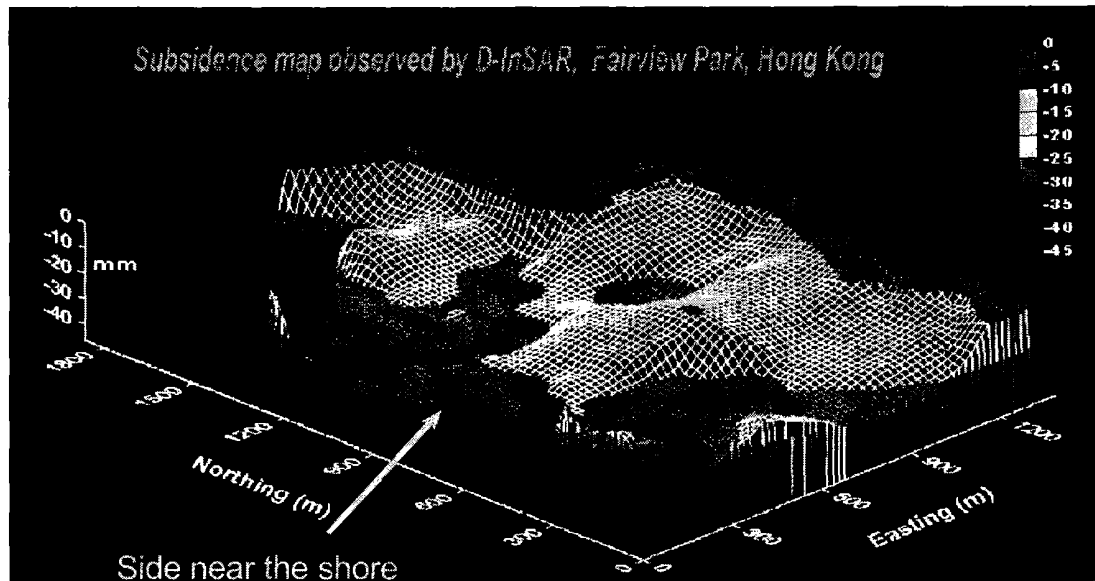


Figure 12: 3D-settlement map over nearly 3 years across the Fairview Park area

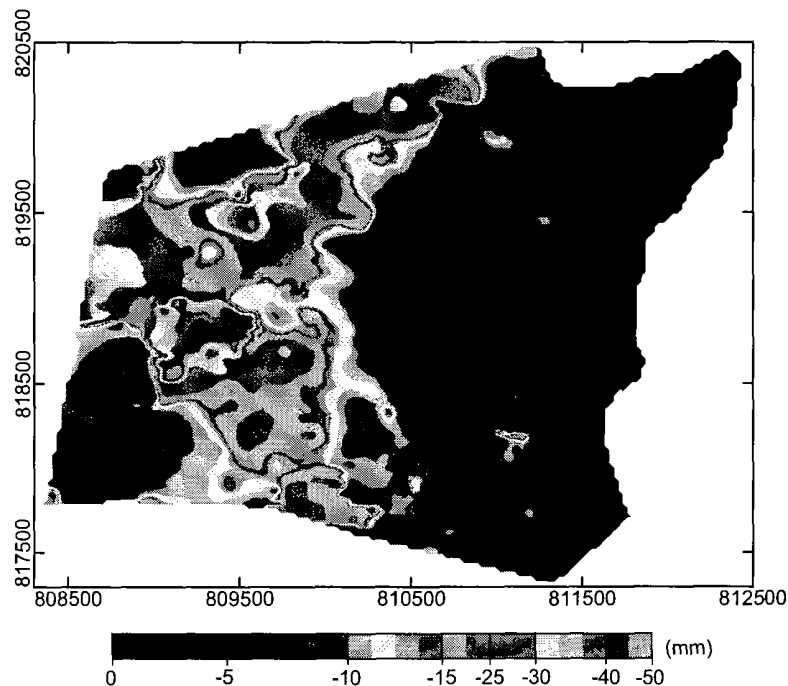


Figure 13: Settlement map of the Chek Lap Kok Airport from the 315-day interferogram.

Settlement at Chek Lap Kok Airport

The interferogram formed between SAR images of 29/12/1998 and 9/11/1999 shows significant differential phase variations over the airport as seen from the deformation map derived from the interferogram (Figure 13). It is very interesting to note that the measured phase values appear to be very smooth across the two excavated islands, Chek Lap Kok and Lam Chau, indicating no or little ground settlement in the areas since they consist of original stable granite. Over the reclaimed areas however, the phase values vary significantly. The InSAR analysis results at this site have been found to be in good agreement with the available settlement survey monitoring records (Ding et al 2004).

Settlement in a Reclamation Site in Tseung Kwan O

The computed relative ground movement at a reclamation site and its neighbouring area in Tseung Kwan O between 24/11/1998 and 29/12/1998 is shown in Figure 14 (right). It should be noted that the results are in relative sense only. The regions with low coherence (<0.15) have been masked out and the average coherence of the remaining area is about 0.4 (Figure 14, left). This level of coherence can, by itself, result in about 3.5 mm errors in the derived deformation field. The InSAR analysis results, which do not match well with the actual settlement pattern, are probably affected by different possible sources of error as described later in this paper.

Ground Movement in Tsing Shan Foothills

The computed relative ground movement in Tsing Shan Foothills, Tuen Mun between 24/11/1998 and 29/12/1998 is shown in Figure 15 (right). The regions with low coherence (<0.15) have been masked out and the average coherence of the remaining area is about 0.48 (Figure 15, left). This level of coherence can cause about 2.8 mm errors in the derived deformation field. The InSAR analysis results do not give a reliable representation of the actual ground movement.

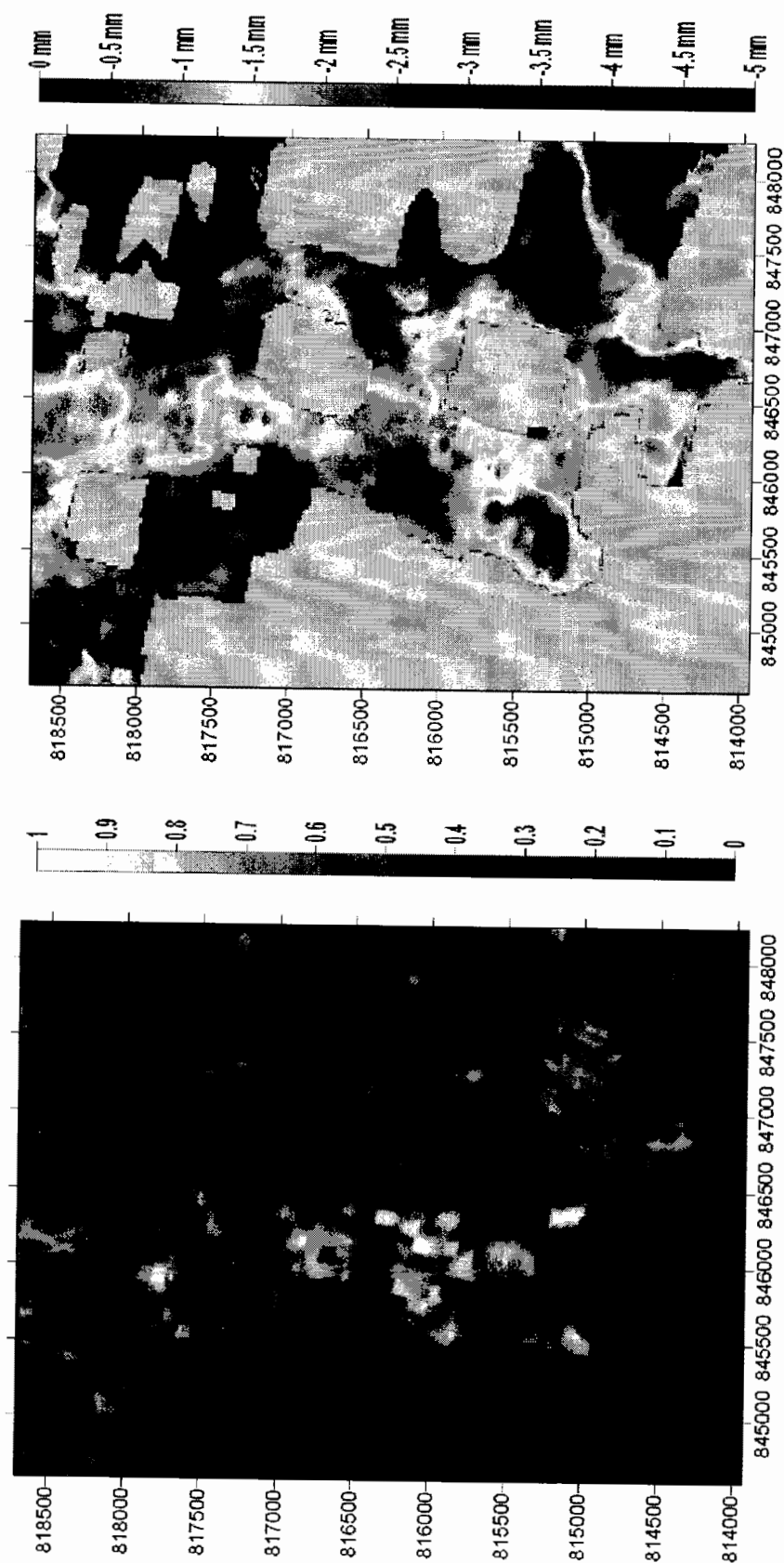


Figure 14: Relative ground movement in a reclamation site and neighbouring area in Tseung Kwan O between 24/11/1998 and 29/12/1998 (right). The coherence image of the whole area is given (left) for reference. The average coherence (excluding masked regions) is about 0.4.

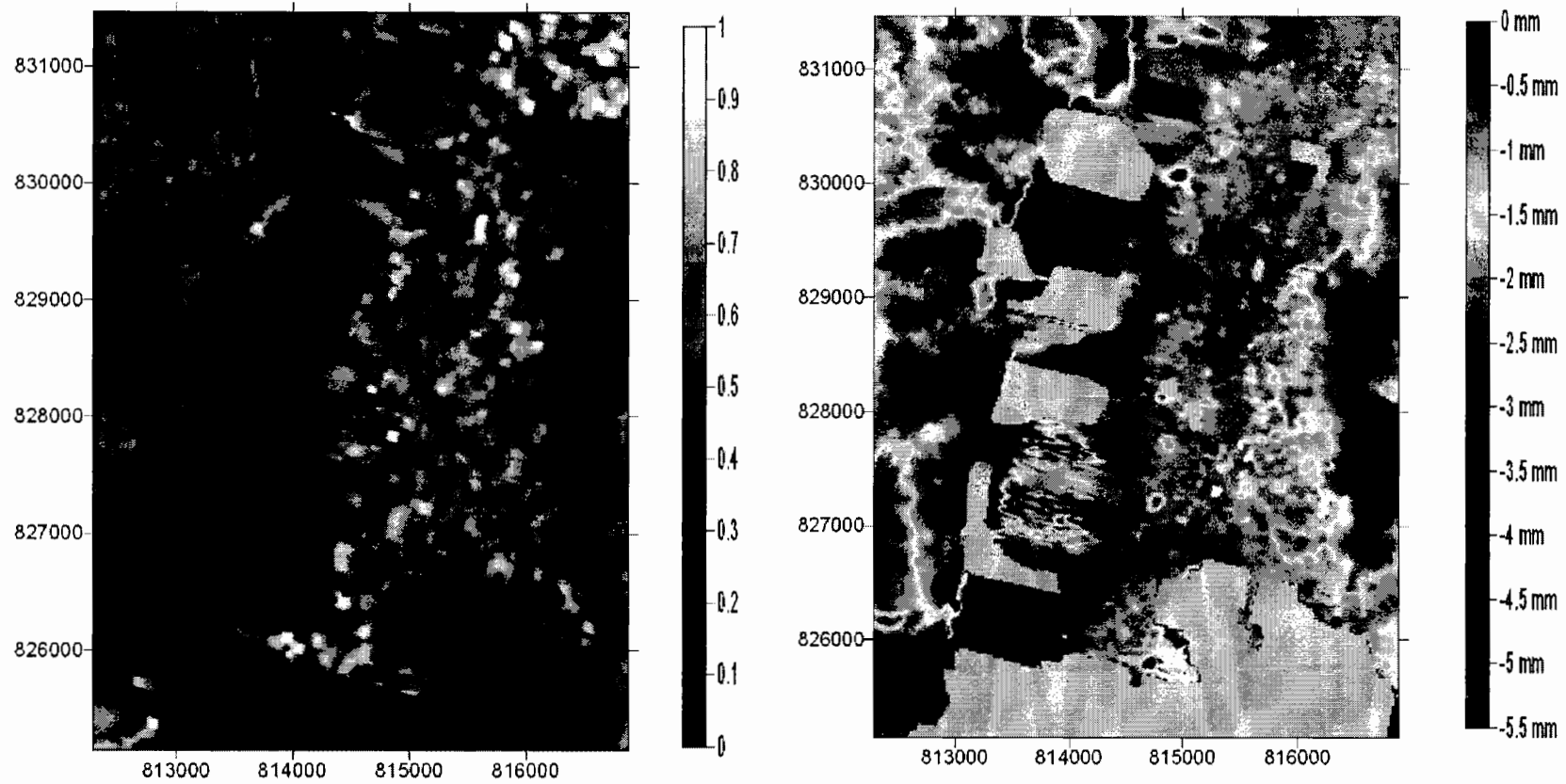


Figure 15: Relative ground movement in Tsing Shan Foothills, Tuen Mun between 24/11/1998 and 29/12/1998 (right). The coherence image is given (left) for reference. The average coherence (excluding masked regions) is about 0.48.

ANALYSIS OF RESULTS

The possible error sources that could limit the accuracy of the measurement results are (1) errors in the orbital parameters, (2) errors in the DEM, (3) phase noise caused by radar decorrelation, and (4) possible atmospheric effects. The orbital errors are negligible for the data set for two reasons: (1) the precise orbit data provided by the Delft University of Technology were used, and (2) the study areas are small (Zebker et al., 1994).

Errors in the DEM can propagate into the differential interferogram and cause phase errors and hence errors in the deformation measurement as seen in Equations (16) and (17). Combining the two equations, we can get

$$\delta_h = \frac{\Delta R}{\cos \theta} = \frac{B_{\perp}}{L \cdot \sin \theta \cdot \cos \theta} \cdot \delta_{DEM} \quad (18)$$

According to this, 5.0 m error in the DEM can lead up to 0.9mm errors in the measured ground movement when using the 35-day pair. The error is therefore also negligible.

The standard deviation of the interferometric phase δ_{ϕ} can be estimated approximately with (Rodriguez and Martin, 1992)

$$\delta_{\phi} = \frac{\sqrt{1 - \gamma^2}}{\gamma \cdot \sqrt{2N}} \quad (19)$$

where γ is the coherence level and N is the window size of the multi-looking operation (5 in this case). For a coherence level of 0.5, the phase standard deviation is about 0.55 radians. This level of phase uncertainty can cause a RMS error of about 2.7 mm in a derived deformation field.

Atmospheric effects can in times become very significant in Hong Kong (e.g., Li et al., 2003; Ding et al., 2004; Li et al., 2004). Further work is required to verify the effects from the atmosphere on the results.

DISCUSSION AND CONCLUSIONS

The following observations on use of InSAR are notable:

- InSAR can be used to detect ground movement over a time interval of several years under good conditions (e.g. good coherence, hard flat surface and insignificant differential atmospheric effect).
- InSAR can be used to detect ground movement at more difficult sites (e.g., vegetated) over short time intervals (say within a month).
- It becomes difficult to get reliable measurement results at difficult sites when the time interval increases to about 3 months.
- It is difficult to measure deformation within a small area (e.g. within tens of meters) and steep slopes due to the limitation of the spatial resolution of the SAR images and geometric distortions.

There is room for further improvement of the InSAR technique and the accuracy of its application in Hong Kong. In particular, the following areas deserve further effort:

- **Establishment of corner reflectors.** Corner reflectors can be used to improve image co-registration and baseline estimation and to calibrate final interferometric results. Hong Kong is a more difficult region in terms of InSAR measurements due to the dense vegetation coverage and the existence of a large number of high-rise buildings. The establishment of some corner reflectors at strategic locations should greatly contribute to the long-term applications of InSAR technology in Hong Kong. There have been successful application examples of corner reflectors in some other regions.
- **Permanent scatter technique.** Permanent scatter (PS) technique is a relatively new interferometric technique that allows InSAR measurements to be carried out for isolated permanent scatters even when the baseline is large for normal InSAR data processing or the overall coherence over an area is low. The method should be suitable for most areas in Hong Kong although it usually requires a large number of SAR images to derive reliable results.
- **Accumulation of future SAR images.** It is useful to plan for acquisition of suitable future ERS and Envisat SAR images that cover Hong Kong. Accumulation of the images will facilitate further studies on use of InSAR in Hong Kong.

REFERENCES:

- Carnec, C., Delacourt, C., 2000. Three years of mining subsidence monitored by SAR interferometry, near Gardanne, France. *Journal of Applied Geophysics*, 43, pp.43-54.
- Ding, X.L., Liu, G.X., Li, Z.W., Li, Z.L., Chen, Y.Q. (2004) Ground Settlement Monitoring in Hong Kong with Satellite SAR Interferometry, *Photogrammetric Engineering and Remote Sensing*, (in press).
- Fielding, E.J., Blom, R.G., and Goldstein, R.M., 1998. Rapid subsidence over oil fields measured by SAR Interferometry, *Geophysical Research Letters*, 25(17): 3215-3218
- Fruneau, B., Achache, J., and Delacourt, C., 1996. Observation and modeling of the Saint-Etienne-de-Tinee landslide using SAR interferometry, *Tectonophysics*, 265:181-190.
- Goldstein, R.M., Zebker, H.A., and Werner C.L. (1988) Satellite radar interferometry: two-dimensional phase unwrapping. *Radio Science*, 23: 713-720.
- Li, Z.W., Ding, X.L., Chen, Y.Q., Liu, G.X., Li, Z.L., Zhu, J.J. (2002) Study of atmospheric effects on InSAR in tropical regions, *Proc. XXII FIG Congress*, Washington, 19-26 April, pp. 1-9 (Paper No. 9).
- Li, Z.W., Ding, X.L., Liu, G.X., Huang, C. (2003) Atmospheric Effects on InSAR Measurements – A Review, *Geomatics Research Australasia*, 79:43-58.
- Li, Z.W., Ding, X.L. and Liu, G.X. (2004) Modeling Atmospheric Effects on InSAR with Meteorological and Continuous GPS Observations: Algorithms and Some Test Results, *Journal of Atmospheric and Solar-Terrestrial Physics* (in press).
- Liu, G.X., 2003. Mapping of Earth Deformations with Satellite SAR interferometry: A study of its accuracy and reliability performance, Ph.D thesis, The Hong Kong Polytechnic University, 229pp.
- Liu, G.X., Ding, X.L., Chen, Y.Q., Li, Z.L. and Li, Z.W. (2001) Monitoring land subsidence at the Chek Lap Kok Airport Using InSAR Technology, *Chinese Science Bulletin*. Vol. 46, No. 21, pp. 1778-1782.
- Liu, G.X., Ding, X.L., Li, Z.L., Li, Z.W., Chen, Y.Q. (2004) Pre- and Co-Seismic Ground Deformations of the 1999 Chi-Chi, Taiwan Earthquake, Measured with SAR Interferometry, *Computers and Geosciences* (in press).

- Massonnet, D., M. Rossi, C. Carmona, F. Adragna, G. Peltzer, K. Feigl, and T. Rabaute(1993), The displacement field of the Landers earthquake mapped by radar interferometry, *Nature*, 364,138~142
- Massonnet, D., Holzer, T. and Vadon, H., 1997. Land Subsidence caused by the East Mesa geothermal field, California, observed using SAR interferometry. *Geophysical Research Letters*, 24(8), 901-904
- Massonnet, D., and Feigl, K. L.(1998), Radar interferometry and its application to changes in the earth's surface, *Review of Geophysics*, 36(4), pp. 441-500.
- Rodriguez, E. and Martin, J.M. (1992) Theory and design of interferometric synthetic aperture radars. *IEEE Proceedings-F*, Vol.139(2), pp.147-159.
- Strozzi, T., Wegmuller, U., Tosi, L., Bitelli, G. and Spreckels, V., 2001. Land subsidence monitoring with differential SAR interferometry, *Photogrammetric Engineering & Remote Sensing*, 67(11):1261-1270.
- Tesauro, M., P. Berardino, R. Lanari, E. Sansosti, G. Fornaro, and G. Franceschetti, 2000. Urban subsidence inside the city of Napoli (Italy) observed by satellite radar interferometry, *Geophysical Research Letters*, 27(13), 1961– 1964.
- Zebker, H.A., Werner, C.L., Rosen, P.A., and Hensley, S., 1994, Accuracy of topographic maps derived from ERS-1 interferometric radar. *IEEE Transactions on Geoscience and Remote Sensing*, Vol.32, 823-836

ACKNOWLEDGEMENTS

The work was supported by the Geotechnical Engineering Office (GEO), the Faculty of Construction and Land Use of the Hong Kong Polytechnic University (under the scheme of Cross-Departmental Collaboration Project) and the Research Grants Council of the HKSAR (Project No.: PolyU5064/02E). This paper is published with the permission of the Head of the Geotechnical Engineering Office and the Director of Civil Engineering of the Government of the Hong Kong Special Administrative Region.

APPLYING MOBILE GIS TECHNOLOGY TO GEOTECHNICAL FIELDWORK

K.C Ng¹, K.S. Fung¹ and W.L. Shum¹

Abstract: In conventional method of conducting fieldwork for geotechnical studies, it is difficult and time-consuming to find an exact location or landmark on site. Also, the staff on site has no means of accessing the wealth of digital information and technology available in the office. These problems significantly constrain the capability, efficiency and quality of geotechnical fieldwork, based on which important decisions affecting public safety have to be made. The Geotechnical Engineering Office of the Civil Engineering Department has recently implemented mobile GIS technology in geotechnical fieldwork. This novel integration of state-of-the-art mobile digital technologies, including GIS, mobile computing, global positioning system, orthoimage and wireless communication, has successfully overcome the conventional fieldwork problems and greatly improved the capability, efficiency and quality of geotechnical fieldwork.

INTRODUCTION

The benefits of accurately recording the correct location of ground features, such as boulders and tension cracks, identified in the field include improved efficiency and quality of the field work and have significant downstream implications on the subsequent analysis and design assumptions for engineering works, e.g. boulder stabilization and landslide mitigation. However, in conventional fieldwork for geotechnical studies, it is often difficult, if not impossible, to find the exact location of ground features, such as loose boulders and tension cracks identified from aerial photograph interpretation, in remote areas or in natural hillsides covered with dense vegetation, particularly where there is no useful geographical features available for reference. The information available on site is generally restricted to hardcopy maps and record notes prepared, without the flexibility of interactively retrieving additional GIS (geographic information system) information available in the office. These constraints significantly hinder the capability, efficiency and quality of geotechnical fieldwork, based on which important public safety decisions are sometime related.

In view of these inherit constraints, the Geotechnical Engineering Office (GEO) of Civil Engineering Department in early 2001 implemented a pilot 'mobile mapping' application to geotechnical engineering fieldwork by extending its GIS capabilities to mobile applications to take advantage of the advances in mobile digital technologies (Figure 1). Since then the system has been fully implemented. Application of the 'mobile mapping system' now covers a range of engineering work, which includes ground investigation, fill management, landslide investigation, squatter studies, geological survey, engineering geological field mapping and geotechnical studies.

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This paper (a) presents the mobile mapping system developed by the GEO and its key components, and (b) discusses the benefits and application of the system and potential areas for improvement.

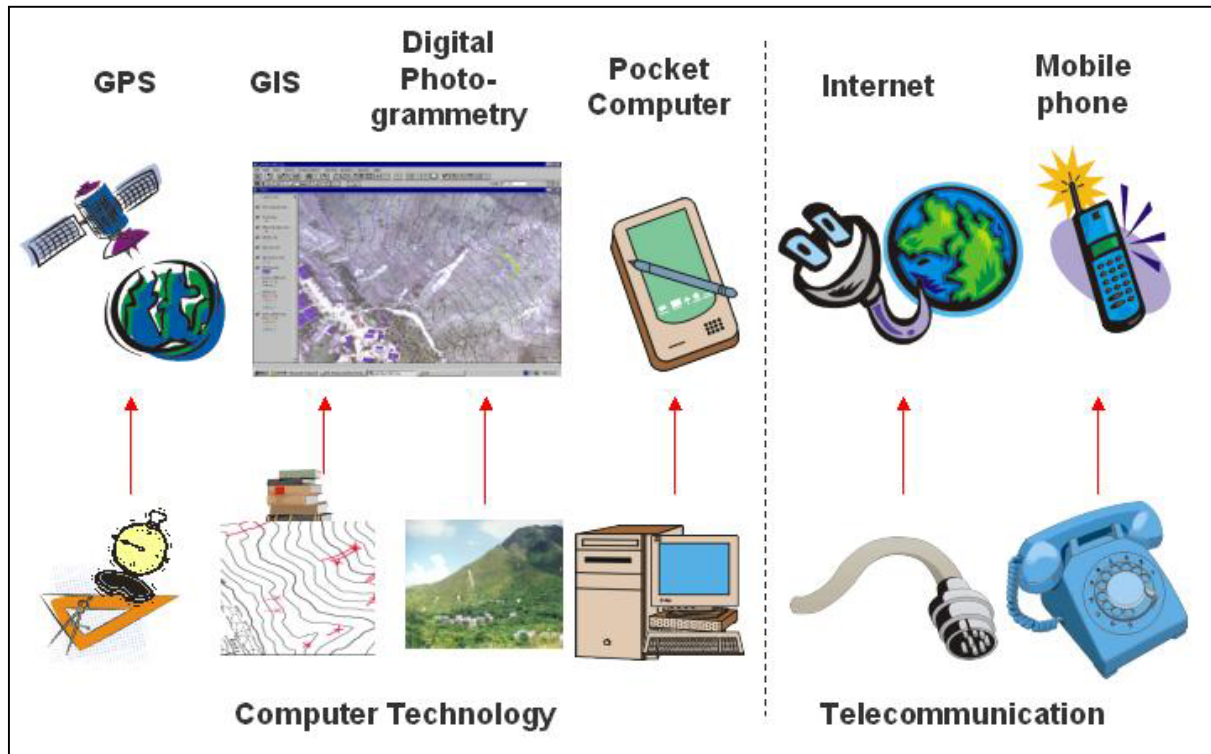


Figure 1 - Extension of digital technologies from desktop to mobile systems

MOBILE MAPPING SYSTEM

The mobile mapping system (Figure 2) integrates state-of-the-art mobile computing devices, global positioning system (GPS) and mobile GIS software into a portable package (Figure 3) for carrying geotechnical engineering fieldwork. The system comprises hand-held (tablet) or pocket computer installed with mobile GIS software for data manipulation, interlinking with GPS device for navigation on-site and Bluetooth-enabled cellular phone for data transfer. Equipment of the mobile mapping system in the GEO is summarized in Table 1. In the office, hardware support stems from servers for Internet access, and workstations to support project preparation. Supporting systems include GIS and digital photogrammetry for data preparation, and specific software for mapping services via the Internet.



Figure 2 - Mobile mapping system.

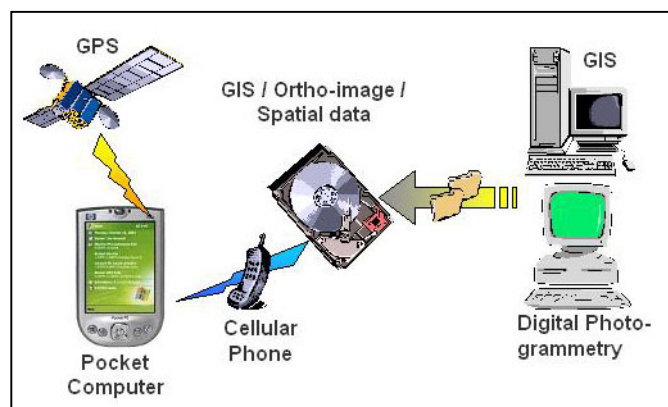


Figure 3 - Inter-relationship of the various components.

Mobile Computing Devices

Hand-held or pocket computer (Figure 4) is used as a platform for digital mapping, location marking, on-site GIS analysis and connection through cellular phones to the office. The key considerations for mobile computing device are portability, power, technical compatibility and display function (Table 1).

Table 1 - Components of the Mobile Mapping System in the GEO

Mobile Computing Device				
Model	Pocket Computer		Hand-held Computer	
	Compaq iPAQ H3650	Compaq iPAQ H3870	Fujitsu Lifebook B-2548	
Size	Pocket Size	Pocket Size	B5 Size	
Weight	170 g	190 g	1400 g	
Display	4.0" Touch Screen Reflective Thin Film Transistor (TFT) LCD	4.0" Touch Screen Reflective Thin Film Transistor (TFT) LCD	10.4" XGA Touch Screen Reflective Thin Film Transistor (TFT) LCD	
Power/Speed	206 MHz Processor, 32 MB SDRAM	206 MHz Processor, 64 MB SDRAM, integrated Bluetooth	500 MHz Processor, 256 MB RAM	
Communications Port/Extensions	Expansion Pack for Bluetooth Card and GPS Plug-in	SD Memory Slot, expansion Pack for GPS Plug-in	USB, card Slots for Bluetooth and GPS Plug-in	
Global Positioning System (GPS)				
Model	GPS (Hand-held)		DGPS	
	Garmin GPS 12XL	Compact GPS	Leica GS 5+	Leica GS 50
Format	GPS	GPS	DGPS	DGPS
Type	Hand-held Display	Plug and Play + Pocket Computer	Bracket-mounted	Stand-alone
Size	Small	Small	Large	Large
Weight	270 g	65 g	660 g	1500 g
Resolution	Within 10 m	Within 5 m	Within 2 m	Within 0.4 m
Telecommunication Device (Cellular Phone)				
Model	Sony Ericsson T68i			
Key Features	Support GPRS up to 40.2 kbps data transmission, equipped with Bluetooth technology, external Plug-in camera			
Mobile GIS Software				
Model	ESRI ArcPad Version 6.0.1			
O/S Compatibility	Window 95/98, NT and 2000 Platforms, Window CE 3.0 and Pocket PC 2002			
Data Format Compatibility	Shapefiles, JPEG, Windows Bitmap, CADRG and MrSID			
Key Functions	Map display with scale and real-time navigation, distance and area measurements, feature and attribute Identification, database access, edit and queries			
Projection Format Compatibility	Geodetic Coordinates, HK Grid 80 and WGS 84 Systems			
Internet Mapping Service				
Model	ESRI ArcIMS Version 3.2			
Key Functions	Provide for distributing, managing mapping and GIS data and service on the Internet, enable users to integrate with local data sources for display, query, and analysis in a Web browser.			

Note: The equipment given in Table 1 is for illustrative purpose only and does not constitute endorsement of the products by the Geotechnical Engineering Office.

Portability is important as geotechnical fieldwork often requires climbing up hillsides and manoeuvre through bushes and trees. Walker and Black (2001) noted the difficulty of carrying field mapping using a laptop weighing over 3 kg. Therefore, there is a need to balance between screen size and weight with respect to hand-held or pocket computer. In order

to access the store of information in the office's databases while on-site, speed and memory capacity is critical for efficient field operations. As regards technical compatibility in terms of communications port/extensions, the ability to provide direct, plug and play connection to the GPS and other accessories is essential. Of special importance is that the display needs to function equally well for inside and outside operations, such that it is able to adjust to the amount of ambient light available.



Figure 4 - Hand-held (left) and pocket (right) computer equipped with GIS software and Bluetooth technology.

Global Positioning System

In the mobile mapping system, the real-time navigation and location information is obtained using Global Positioning System (GPS) methods. The United States GPS is a worldwide system established from a constellation of 24 satellites rotating around the earth in precisely known orbits at an altitude of about 20,200 km, specifically designed to aid positioning and navigation (USACE 2003). They transmit signal information to the ground and GPS receivers take them as reference points to calculate positions accurate to within a few metres (USACE 2003). The GPS receiver can be of either hand-held or plug-in type (Figure 5). Prior to May 2001, it was not possible to obtain metre level accuracy due to United States Government's intentional degradation (also referred to as 'Selective Availability') of GPS signals (NOAA 2001). Discontinuing the use of signal degradation enables pinpointing locations up to ten times more accurate than previously possible, i.e. within 10 m resolution using hand-held GPS (Table 1). The GPS signals are available 24 hours a day and free of charge.



Figure 5 - Hand-held (left) and compact flash (right) GPS receivers.

GPS measurements from satellites alone are potentially subject to deviations/errors of the individual satellite signal received, such as uncertainties in the satellite orbits, variations in atmospheric conditions, receiver errors, and multi-path errors (Lillesand and Kiefer 2000). Differential GPS (DGPS) measurement methods, which involve the synchronisation of two

receivers, one from a stationary ground station and another receiving signals from moving satellites, can be used to eliminate and compensate for the errors. The positional measurements at the ground station are used to refine the position measured by the GPS receivers. In Hong Kong, Marine Department maintains a ground station at Kau Yi Chau for public access at no cost to users. Using the portable form of DGPS (Figure 6), it is possible to make measurements (x, y and z co-ordinates) well within 1 m resolution (Table 1).



Figure 6 - Two portable types of DGPS receivers.

Geographic Information System

Longley et al. (2001) noted many definitions of GIS suggested over the years, depending on the end users. In essence, GIS is a decision support system involving the storage, manipulation, analysis and presentation of spatially referenced information in a problem solving environments. Since early 1990s, the GEO has been developing and applying GIS, for managing its spatial data and solving geotechnical engineering problems. In the last few years, some new initiatives have been implemented by taking advantage of the advances in GIS and digital technologies. In early 2001, the emergence of mobile GIS has permitted extension of GIS capabilities to application in geotechnical engineering fieldwork.



Figure 7 - Pre-loading of data into a pocket computer.

The heart of the mobile mapping system is the Geographic Information System (GIS) operating on a mobile computing platform (referred to as 'mobile GIS'). Spatial data such as digital topographical maps, ortho-rectified aerial photographs, engineering and geological GIS data can be pre-loaded into the pocket computer prior to fieldworks (Figure 7), or downloaded on-site through the Internet gateway. Field information on the features of interest can be instantly recorded on-site as GIS data and on customized form (Figure 8) as appropriate, and their corresponding



Figure 8 - Field information can be recorded as GIS data (left) and on customized form (right).

GPS locations can be registered in the mobile GIS (Figure 9). The mobile GIS also enables real-time navigation in association with GPS and some simple GIS operations in the field such as distance and area measurements, and spatial queries. Software for mapping services via the Internet has made the wealth of GIS data available in the office becomes accessible in the field through wireless communication, and for integration with all data sources for GIS operations in a Web browser.

Orthoimage

Vertical aerial photographs are geometrically distorted due to tilting of camera, fluctuation of flight altitude and effects of topographical relief. Removal of the resulting image displacement and scale variation is done through a process called orthographical rectification (Jensen 2000), using a digital photogrammetric workstation. The product is an ortho-rectified digital image (orthoimage), showing the true position and scale of terrain and ground features, which can be used alone or be integrated with other data within a geographic information system.

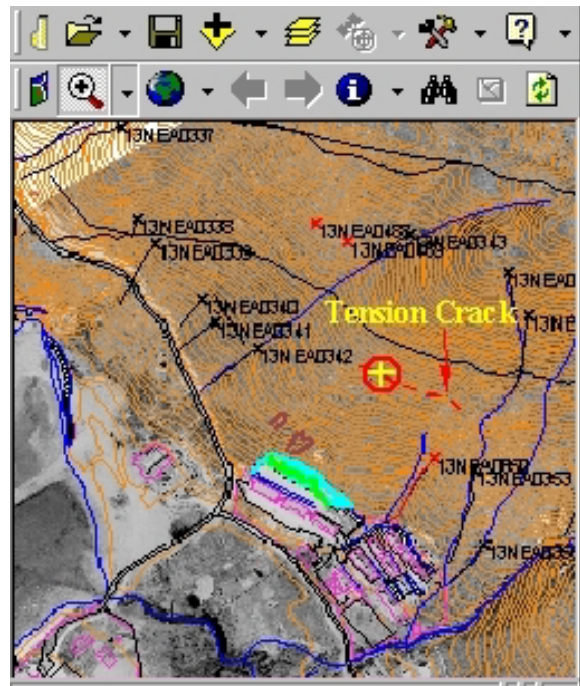


Figure 9 - Orthoimage, topographical map, ground features and GPS location displayed on a pocket computer.

In the natural terrain hazard study for Tsing Shan Foothills, field mapping was carried out on orthoimages coupled with 1:1000-scale topographical base plans (Ruse et al. 2002). The scaled orthoimage of the landslides proved especially invaluable during field mapping as they provided accurate location for precise mapping of landslide scars and associated features (Figure 10), not simply cartographic representations of those features as in the conventional topographical plans. Incorporating an orthoimage with a topographical base plan on a mobile GIS platform can greatly enhance the mapping capability as topographical details, terrain information and ground features are on hand for reference (Figures 9, 10).

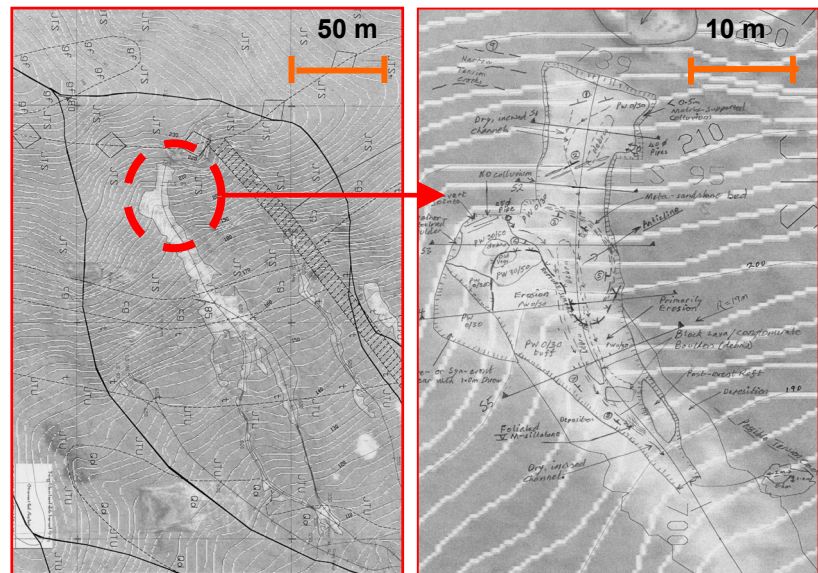


Figure 10 - Mapping of a landslide scar on an orthoimage coupled with a contour plan.

Wireless Communication

Wireless communication is a key component for establishing linkage between the office and the field. The cellular phone, using General Packet Radio Service (GPRS) for high-speed wireless data transmission via the Internet, serves as a modem for wireless communication. The always-connected capability of GPRS allows continuous linkage with the servers in the office such that instant data retrieval can be obtained. The cellular phone enabled with Bluetooth technology is then wirelessly integrated with the hand-held or pocket computer of the mobile mapping system (Figures 2, 3). As there is no line-of-sight requirement for connection, it is much more convenient to operate in the field with the Bluetooth technology as compared to the traditional infra-red connection.

DISCUSSION

The development of the pilot mobile mapping system by the GEO in early 2001 was at an opportune time when a number of coincidental events came about, namely (a) surfacing of commercial mobile GIS software, (b) availability of pocket computer capable of operating GIS software, (c) emergence of software for Internet mapping service, (d) maturity of Bluetooth technology for wireless communication, and (e) discontinuation of 'Selective Availability' of GPS signals by the United States Government. Furthermore, the technical compatibility of the mobile computing devices allows a direct, plug and play system without any need for customization. The integrated system provides on-site navigation guided by GPS and orthoimages on a mobile GIS platform, which has revolutionized the way of performing geotechnical fieldwork.

Benefits of Mobile Technology

A vast number of benefits have emerged from the development of the mobile mapping system. The mobile mapping system can be used for accurate search and positioning of ground features, such as landslide scars, tension cracks, signs of distress, boulders and ground investigation stations, and data transmission between office-field communication can be done through wireless connection. As such, geotechnical fieldwork quality and efficiency is greatly improved. Specifically, the system has resulted in the following benefits in various types of geotechnical works:

- (a) Geotechnical fieldwork efficiency and cost-effectiveness is improved by the integration of location positioning and surveying, the functionality of GIS data, geotechnical mapping techniques, and mobile computing and data processing in one hand-held unit.
- (b) The capability and quality of geotechnical fieldwork by direct connection to office-based GIS data is greatly enhanced.
- (c) Data transmission and office-field communication is improved, facilitating decision-making that affects public safety, especially with emergency situations, such as a serious landslide.

The system also serves as a management system to capture work data and job information that is field orientated to facilitate the compilation of management information and statistical reports. This administrative function will save the staff time and effort in capturing primary data using paper-based forms at site. Hand-held or pocket computers tailored with electronic forms can be used to facilitate data capturing.

Application of Mobile Technology

Most geotechnical projects, either regional or site-specific nature, deal with some form of spatial data. To capitalize the benefits offered by the mobile mapping technique, the GEO has applied it to the following broad geotechnical functions:

- (a) Landslide studies and assessment of follow-up actions after landslides,
- (b) Natural terrain hazard assessments, geological survey and engineering geological mapping,
- (c) Geotechnical assessment for safety clearance of squatters,
- (d) Fieldwork for geotechnical studies and fill management related work, and
- (e) Setting out of ground investigation stations.

Recently, mobile GIS incorporating the use of GPS has also been employed in other aspects of Government's work in the SAR. Some notable examples are Marine Department's system of locating, placing and checking of buoys, Lands Department's method of locating control points for land surveying, and Highways Department's scheme of locating sites of road defects.

Elsewhere, Vivoni (2002) has presented a mobile GIS/GPS module integrated with water quality monitoring device for environmental field data collection and Koostra et al. (2003) have demonstrated a integrated suite of mobile GIS and GPS for field data collection for precision agriculture applications. It has been estimated that 80% of all data has a spatial component (Burrough and McConnell 1998); therefore, limit of mobile mapping application is to the imagination.

Areas for Improvement

Digital technology is not a static subject and the mobile mapping system is of no exception as advances are being made on a routine basis. For example, the recently available 'Third Generation Wireless System' (3G) transmits 1 MB of data in matter of seconds as compared to couple of minutes using GPRS and in multimedia format in a mobile environment. Various components of the system could be enhanced to improve its capability and user friendliness for geotechnical application. Some areas for improvement have been identified as follows:

- (a) Hand-held or pocket computer – (i) suitable for all weather conditions, (ii) more efficient power management and longer battery life, (iii) higher processing speed, and (iv) voice-enabled operation.
- (b) GPS – (i) the United States is planning for development of a new generation GPS satellites that could provide much higher horizontal and vertical accuracy than those that have been in use for quite some time, and (ii) real-time access to Lands Department's ground stations for DGPS applications.
- (c) Telecommunication – (i) improved coverage of the 3G wireless system for the SAR, and (ii) enhancement of the system to its optimum transmission speed.

CONCLUSIONS

The geotechnical engineering practices have been able to greatly advance as a result of the successful integration of the mobile mapping system. As the capabilities of engineering practices increase, the work production increases in quality and efficiency. The entire workflow procedures process become faster and more accurate as the paper work and data conversion are done electronically. Besides the obvious benefits of enhancing fieldwork capability, the mobile application also enables capturing work data to facilitate the compilation of management information that is field orientated and hence saves time and effort in capturing primary data using paper-based forms at site.

REFERENCES

- Burrough, P.A., and McConnell, R.A. (1998). *Principles of Geographical Information Systems*. Oxford University Press, London.
- Jensen, J.R. (2000). *Remote Sensing of the Environment – An Earth Resource Perspective*. Prentice Hall, New Jersey.
- Lillesand, T.M., and Kiefer, R.W. (2000). *Remote Sensing and Image Interpretation*. 4th Edition, John Wiley & Sons, Inc., New York.
- Longley, P.A., Goodchild, M.F., Maguire, D.J., and Rhind, D.W. (2001). *Geographic Information Systems and Science*. John Wiley & Sons, New York.
- Kooststra, B.K., Stombaugh, T.S., and Dowdy T.C. (2003). “Development of a precision agriculture tool suite using ArcPad.” ESRI User Conference Proceedings, 23rd Annual ESRI International User conference, July 7-11, 2003, San Diego, California, U.S.A., 1-14 (in CD-ROM).
- Ruse, M.E., Waring, D.P., Kaldy, A., Chan, S.K., and Ng, K.C. (2002). “Initiation and runout characteristics of a swarm of 121 landslides in the Tsing Shan Foothills, Hong Kong.” *Proceedings of the Conference: Natural Terrain – A Constraint to Development?*, The Institution of Mining and Metallurgy (Hong Kong Branch), 14 November 2002, Hong Kong, 77-87.
- NOAA (National Oceanic and Atmospheric Administration). (2001). *Statement by the President regarding the United States' Decision to Stop Degrading Global Positioning System Accuracy*. NOAA Web Site <www.ngs.noaa.gov>.
- USACE (United States Army Corps of Engineers). (2003). *Engineering and Design – NAVSTAR Global Positioning System Surveying*. United States Army Corps of Engineers Internet Publishing Group, Publication No. EM 1110-1-1003, 1 July 2003.
- Vivoni, E. (2002). “Development of software tools for environmental field studies using ArcPad.” Abstracts of Conference Proceedings, 2002 ESRI International User Conference, July 8-12, 2002, San Diego, California, U.S.A., 7-8.
- Walker, J.D., and Black, R.A. (2000). “Teaching field geology with laptop computers and geographic information systems brings digital mapping to the outcrop”. *Geotimes*, 45(11), 28-31.

ACKNOWLEDGEMENTS

This paper is published with the permission of the Head of the Geotechnical Engineering Office and the Director of Civil Engineering of the Government of the Hong Kong Special Administrative Region.

A CIRCA 0.5-MILLION YEAR GEOLOGICAL MODEL FOR GEOTECHNICAL ENGINEERING IN HONG KONG

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Abstract: A circa 0.5-million year geological model covering the last five interglacial-glacial cycles in the Quaternary with geotechnical engineering applications in Hong Kong is proposed. High sea levels during interglacial periods can be recognized by the presence of up to five layers of marine deposits (M1 to M5 units) while low sea levels during glacial periods can be recognized either by the presence of up to five layers of terrestrial deposits (T1 to T5 units) and/or palaeosols (referred to as palaeo-desiccated crusts) formed by the sub-aerial exposure of marine deposits. The model is supported by sedimentological evidence, palaeontological evidence, engineering properties, age dating, and, geological correlation including with the Vostok ice core in Antarctica. Based on the model, a better understanding of engineering properties obtained from field and laboratory testing is possible. Both the depositional environment and the post-depositional environmental changes are found to be important in determining the engineering properties of transported soils. In the present study, selected engineering properties for marine deposits (M1 to M3 units) from three coastal engineering projects – the new Hong Kong International Airport, Penny's Bay and Southeast Kowloon have been analyzed using statistical methods. The results are found to support the model.

INTRODUCTION

The Quaternary spanning the last 2.4 million years of geological time has emerged as the most important and most intensively studied period because of the relevance to global environmental change and future human survival (Williams et al. 1998). In 1928, the International Union for Quaternary Research (INQUA) was established to coordinate multi-disciplinary research work for the Quaternary period throughout the world. New research findings of international significance are disseminated mainly through a number of journals including Quaternary Research, Quaternary Science Review, Quaternary International, Journal of Quaternary Science and Holocene. Additionally, findings with major scientific impacts also appear frequently in the two general science journals Nature and Science. These journals are useful for anyone wishing to keep abreast with the latest development in Quaternary science.

Terzaghi (1927) in Skempton (1944) was probably the first to note the relationship between Quaternary environmental changes and the engineering properties of transported soils. The importance of the topic to geotechnical engineering was highlighted notably by Fookes (1991). However, it was not until Fookes (1997) at the First Glossop Lecture that the recognition of geological models became popular. At the Third Glossop Lecture, Chandler (2000) introduced the geotechnical cycle for clay sediments in depositional basins as a soil behavior model taking into account deposition, consolidation/compaction, diagenesis, tectonic disturbance, weathering and erosion. In Hong Kong, the importance of Quaternary

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geology in comparison to pre-Quaternary geology in civil engineering construction is reflected in the amount of treatment devoted to the period in the two companion publications of the Hong Kong Geological Survey. The Pre-Quaternary Geology of Hong Kong volume (Sewell et al. 2000) has 181 pages while the Quaternary Geology of Hong Kong volume (Fyfe et al. 2000) has 209 pages.

In the present study, a geological model covering circa 0.5 million years supported by engineering properties of transported soils in Hong Kong and other evidence is proposed. Ground investigation data from three coastal engineering projects – the New Hong Kong International Airport (HKA), Penny's Bay (PB) and Southeast Kowloon (SK) were used for statistical analysis to test the applications of the model.

Why is a Geological Model Needed?

For studying transported soils in Hong Kong, a 'well-proven' geological model is needed for assisting ground investigation as well as for engineering design and construction. This is particularly important for the coastal areas of Hong Kong because they have been affected by Quaternary sea-level changes during interglacial and glacial periods. During ground investigation, a geological model is needed for applications ranging from the logging of drill-holes, the ground-truthing of geophysical investigations including offshore seismic profiling, and the interpretation of engineering properties obtained from field and laboratory testing. What is frequently ignored but important is the geological history of the soils since their deposition including the impact of post-depositional sub-aerial exposure on marine deposits. Without a geological model taking into account environmental changes, it would not be possible to understand the range of physical and chemical properties in the soil types. The model should also help to reduce errors in drill-hole logging.

Background

Some engineering applications of Quaternary sea-level changes in Hong Kong have already been presented by Yim and Choy (2000). A geological model in agreement with global sea-level changes covering a period of circa 0.5 million years based mainly on sedimentological and engineering investigations of offshore drill-hole samples supported by age dating was proposed (Yim 1994; Yim and Tovey 1995; Yim and Choy 2000; Tovey and Yim 2002). This model is also supported by findings made on the Vostok ice core in Antarctica (Petit et al. 1999) which is currently the highest resolution record covering circa 0.5 million years of past earth history.

THE GEOLOGICAL MODEL

Table 1 provides a summary of the geological model proposed. In this model, a maximum of up to five marine units (M1 to M5) and five terrestrial units (T1 to T5) separated by hiatuses representing five interglacial-glacial cycles can be recognized. The main characteristics of the individual units are:

- (1) M1 unit of Holocene or Marine Isotope Stage (MIS) 1 age –

This is usually a very soft to soft, predominantly grey clayey silt with complete shells and shell fragments. The grey colour is caused by the presence of pyrite which was formed by bacterial activity on the seafloor (Bernier 1984).

- (2) T1 unit of last glacial to pre-last interglacial or MIS 2-4 age –

This is usually a loose to firm, gravelly sand of whitish to yellowish colour. The origin is predominantly alluvial.

- (3) M2 unit of last interglacial or MIS 5 age –

This unit can be divided into two parts. The upper part is usually a firm to stiff, sandy clayey silt showing mottling with colours ranging from white, yellow, orange, red, brown and grey. It is identified to be a palaeosol, referred to as a palaeo-desiccated crust formed by the post-depositional sub-aerial exposure of the last interglacial marine deposits (Tovey and Yim 2002). The presence of a palaeosol is supported by evidence including the presence of lithified plant rootlets, the absence or the rarity of calcareous marine fossils (Yim 1992; Yim 1994; Yim and Tovey 1995; Tovey and Yim 2002) and a sharp increase in magnetic susceptibility (Yim et al. 2004). The lower part is usually a soft to firm clayey silt of predominantly grey colour with occasional shell fragments.

(4) T2 unit of second last glacial or MIS 6 age –

This is usually a loose to firm, gravelly sand of whitish to yellowish colour. The origin is predominantly alluvial.

Table 1 The geological model for Hong Kong after Yim and Choy (2000).

Stratigraphic unit+	Estimated age (yrs Before Present)	Marine isotope age	Age	Maximum thickness (m)
M1	< 8,100	1	Holocene/post glacial	21.5
T1	8,100 – 70,000	2-4	Last glacial	6.5
M2	90,000 – 140,000	5	Last interglacial	15.7
T2	150,000 – 180,000	6	2 nd last glacial	9.5
M3	190,000 – 240,000	7	2 nd last interglacial	12
T3	250,000 – 300,000	8	3 rd last glacial	7.3
M4	310,000 – 340,000	9	3 rd last interglacial	14.1
T4	350,000 – 370,000	10	4 th last glacial	6
M5	380,000 – 420,000	11	4 th last interglacial	3.5
T5	> 440,000	12	pre-4 th last interglacial	7

+ M – marine; T – terrestrial.

(5) M3 unit of second last interglacial or MIS 7 age –

Like the M2 unit, this unit can also be divided into two parts. The upper part is usually a firm to very stiff, sandy clayey silt showing mottling with colours ranging from white, yellow, orange, red, brown and grey. It is also identified to be a palaeosol and is referred to as a palaeo-desiccated crust formed by the post-depositional sub-aerial exposure of the second last interglacial marine deposits. The existence of the palaeosol is supported by the same evidence found for the M2 unit. The lower part is usually a soft to firm clayey silt of predominantly grey colour with occasional shell fragments.

(6) T3 unit of third last glacial or MIS 8 age –

This is usually a loose to firm, gravelly sand of whitish to yellowish colour. The origin is predominantly alluvial.

(7) M4 unit of third last interglacial or MIS 9 age –

Like the M2 and M3 units, this unit can also be divided into two parts. The upper part is usually a firm to very stiff, sandy clayey silt showing mottling with colours ranging from white, yellow, orange, red, brown and grey. It is also identified to be a palaeosol and is referred to as a palaeo-desiccated crust formed by the post-depositional sub-aerial exposure of the third last interglacial marine deposits. The palaeosol origin is supported by the same evidence found for the M2 and M3 units. The lower part is usually a firm to stiff clayey silt of predominantly grey colour with occasional shell fragments.

(8) T4 unit of fourth last glacial or MIS 10 age –

This is usually a loose to firm, gravelly sand of whitish to yellowish colour. The origin is predominantly alluvial.

(9) M5 unit of fourth last interglacial or MIS 11 age –

Like the M2 and M3 units, this unit can also be divided into two parts. The upper part is usually a firm to very stiff, sandy clayey silt showing mottling with colours ranging from white, yellow, orange, red, brown and grey. It is identified to be a palaeosol and is referred to as a palaeo-desiccated crust formed by the post-depositional sub-aerial exposure of the fourth last interglacial marine deposits. The palaeosol origin is supported by the same evidence found for the M2, M3 and M4 units. The lower part is usually a firm to stiff clayey silt of predominantly grey colour with occasional shell fragments.

(10) T5 unit of fifth last glacial or MIS 12 age –

This is usually a loose to firm, gravelly sand of whitish to yellowish colour. The origin is predominantly alluvial.

The complete succession with all five marine units was found in the West Lamma Channel (Yim et al. 1997; Yim and Li 2000). Summaries of selected features useful for distinguishing between the marine and terrestrial units can be found in Yim (1992) and Yim (1994). The diagnostic features for all of the marine and terrestrial units including age dating results can be found in Yim and Choy (2000).

TESTING THE GEOLOGICAL MODEL

A location map of the three localities chosen for testing the geological model is shown in Figure 1. These localities were chosen because of their 'large' ground investigation databases including extensive field and laboratory testing. For HKA, 100 pre-reclamation drill-holes and 93 pre-reclamation cone penetration test sites were selected. For PB, 98 pre-reclamation drill-holes (including 45 continuously sampled vibrocores) and 13 pre-reclamation cone penetration test sites were selected. For SK, 82 pre-reclamation drill-holes (including 55 continuously sampled vibrocores) and 32 pre-reclamation cone penetration test sites were selected.

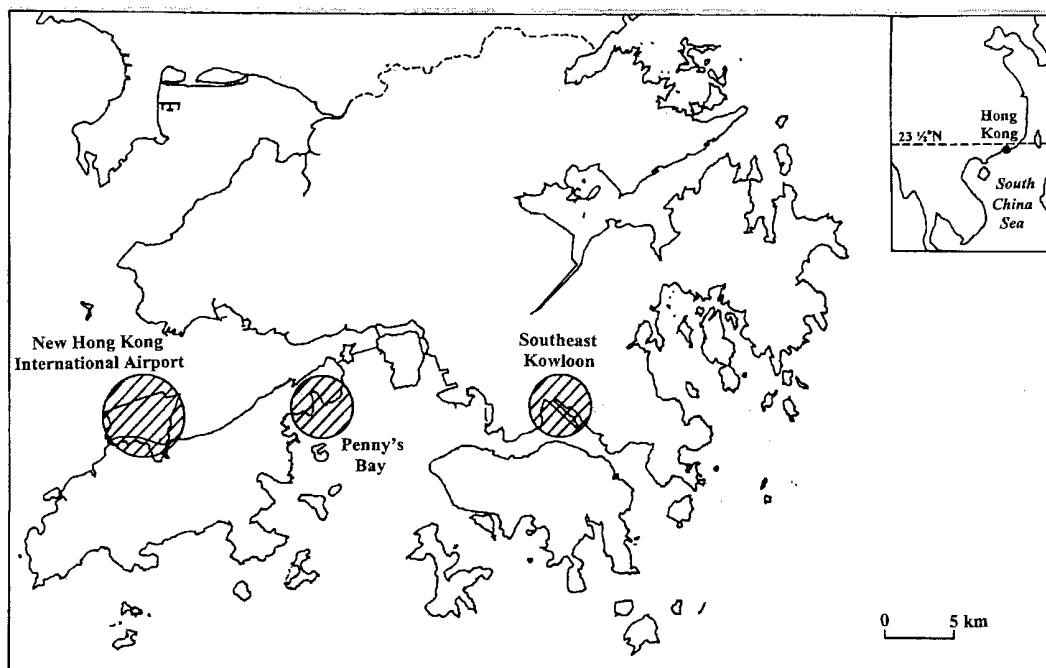


Figure 1. Location map of the three coastal engineering projects selected for study.

The logsheets of all drill-holes selected were reinterpreted aided by the examination of core samples, core photographs and cone penetration testing profiles leading to the recognition of the marine and terrestrial units described in the previous section. Engineering properties determined through field and laboratory testing on drill-hole samples from the M1, M2 and M3 units including moisture content, initial void ratio, bulk density, liquid limit, plastic limit, liquidity index, overconsolidation ratio and torvane shear strength were then selected for statistical analysis using mean, range, standard deviation, coefficient of variation and correlation coefficient. This was carried out using the software STATISTICA which also takes into account missing values of engineering properties.

A total of 247 samples from one continuously sampled rotary drill-hole down to a depth of 39.52 m below present seabed or -48.32 m below Principal Datum from the New Hong Kong International Airport site was selected for moisture content measurements. The results obtained which confirmed the presence of the M1, M2, M3 and M4 units with palaeo-desiccated crusts present in the upper part of the pre-M1 units can be found in Yim et al. (2002b). Two continuously sampled vibrocores from the PB site and eight continuously sampled vibrocores from the SK site were selected for magnetic susceptibility measurements to identify palaeosols formed by sub-aerial exposure during the last glacial period. The results can be found in Yim et al. (2004) and Davis et al. (1999) respectively.

RESULTS

Table 1 provides a summary of selected engineering properties of the Holocene M1 unit in the three study areas. This unit has the characteristics of materials which have never been sub-aerially exposed but the upper part is affected by anthropogenic disturbance to different degrees through shipping activities including trawling. The SK site is additionally affected by dredging, sewage discharge and stormwater discharge. The characteristics found include:

Table 1 Statistical summary of selected engineering properties of the Holocene M1 unit in the three study areas.

Property	Locality	No. of samples	Range	Mean	Standard deviation	Coefficient of variation (%)
Moisture content (%)	A	229	21.7 - 127.3	93.3	17.3	18.5
	B	54	61.8 - 124.2	93.7	11.6	12.4
	C	43	23 - 100	53.2	15.6	29.3
Initial void ratio	A	85	1.28 - 3.37	2.63	0.46	17.5
	B	32	1.81 - 3.09	2.57	0.17	6.1
	C	40	0.57 - 3.64	1.43	0.53	37
Bulk density (Mg/m ³)	A	227	1.119 - 2.644	1.469	0.15	10.2
	B	53	1.35 - 1.549	1.451	0.051	3.5
	C	40	1.36 - 2.063	1.712	0.143	8.4
Liquid limit (%)	A	193	37 - 107	78.2	13.9	17.8
	B	53	32 - 90	70.6	11.6	16.4
	C	43	34 - 80	54.2	11	20.3
Plastic limit (%)	A	193	19 - 64	31.8	6.1	19.2
	B	53	19 - 54	31.4	6.9	22
	C	43	18 - 38	24.9	4.4	17.7
Liquidity index	A	193	-1.47 - 2.42	1.19	0.67	56.3
	B	53	1 - 5.51	1.72	0.71	41.3
	C	43	0.34 - 2.78	1.26	0.45	35.7
Overconsolidation ratio	A	76	0.9 - 4.4	1.8	0.6	33.3
Torvane shear strength (kPa)	A	243	2 - 70	10.8	8.5	78.7
	B	171	0 - 18	6.3	3.5	55

A- New Hong Kong Airport; B- Penny's Bay; C- Southeast Kowloon.

Coefficient of variation = Standard deviation x 100 / Mean %.

- (1) High moisture content with mean values exceeding 90 % at the HKA and PB sites. The lower value at the SK site is accounted for by the presence of a significant sand and gravel fraction either caused by the construction of the Kai Tak Airport and/or dredging.
- (2) High mean initial void ratios exceeding 2.5 are found at the HKA and PB sites. The low mean value found at the SK site is accompanied by the highest coefficient of variation and is also indicative of the sand and gravel fraction introduced through anthropogenic disturbance.
- (3) The mean bulk density is highest at the SK site followed by the HKA site and the PB site. This is accounted for by anthropogenic disturbance at the SK site (Davis et al. 1999) and the influence of escaping gas at the PB site (Premchitt et al. 1992; Langford et al. 1995; Yim et al. 2002a).
- (4) Both the mean values for liquid limit and plastic limit at the HKA and PB sites are similar while the SK site is lower. The presence of significant amounts of sand and gravel fraction at the SK site is the reason for the lower cohesion.
- (5) The mean liquidity indices exceed 1 with PB (1.72) the highest. At the HKA site, negative values and values below 1 indicate overconsolidation which may be caused by the presence of clasts detached from the palaeo-desiccated crust of the M2 unit (Tovey and Yim 2002). This is also supported by the overconsolidation ratio found at the site.
- (6) The lower mean torvane shear strength at the PB site compared to the HKA site is also accounted for by the influence of escaping gas.

Table 2 shows the correlation coefficients between selected engineering properties in the Holocene M1 unit of the three study areas. The strongest positive correlation is found at the

Table 2 Correlation coefficients between selected engineering properties in the Holocene M1 unit of the three study areas. Statistically insignificant correlation coefficients are marked by asterisks.

	Locality	Moisture content	Initial void ratio	Bulk density	Liquid limit	Plastic limit	Torvane shear strength	<i>In situ</i> vertical stress	Maximum past pressure
Moisture content	A	1.00	0.99	-0.95	0.80	0.68	-0.5	-0.62	-0.59
	B		0.83	-0.56	0.39	0.34*	-	-	-
	C		0.86	-0.85	0.82	0.67	-	-	-
Initial void ratio	A		1.00	-0.96	0.78	0.67	-0.61	-0.62	-0.58
	B			-0.74	0.51	0.38	-	-	-
	C			-0.92	0.68	0.65	-	-	-
Bulk density	A			1.00	-0.82	-0.72	0.55	0.58	0.58
	B				-0.26*	-0.23	-	-	-
	C				-0.71	-0.57	-	-	-
Liquid limit	A				1.00	0.79	-0.25	-0.21*	-0.20*
	B					0.44	-	-	-
	C					0.83	-	-	-
Plastic limit	A					1.00	-0.28	-0.18*	-0.19*
	B						-	-	-
	C						-	-	-
Torvane shear strength	A						1.00	0.68	0.78
	B							-	-
	C							-	-
<i>In situ</i> vertical stress	A							1.00	0.80
	B								-
	C								-
Maximum past pressure	A								1.00
	B								
	C								

A- New Hong Kong Airport (67 samples); B- Penny's Bay (32 samples); Southeast Kowloon (40 samples).

HKA site followed in order by the SK and PB sites. At the HKA site, the strongest positive correlation is between moisture content and initial void ratio (0.99) followed jointly by

moisture content and liquid limit, and *in situ* vertical stress and maximum past pressure (both 0.80). The strongest negative correlation is between initial void ratio and bulk density (-0.96) and between moisture content and bulk density (-0.95). This is explained by conditions at the HKA site where predominantly silt size have been reworked by moderately strong tidal currents. The weaker correlation at the SK and PB sites may be explained by anthropogenic disturbances and escaping gas formed by the bacterial breakdown of organic matter from the underlying pre-M1 units (Yim et al. 2002a).

Table 3 provides a summary of selected engineering properties of the last interglacial M2 unit in the three study areas. At the HKA and SK sites, the engineering properties for samples from the palaeo-desiccated crust and from below the palaeo-desiccated crust are grouped separately to facilitate comparison. The PB site samples are undifferentiated because a weakly developed palaeo-desiccated crust is present. The characteristics found include:

Table 3 Statistical summary of selected engineering properties of the last interglacial M2 unit in the three study areas.

Property	Locality	No. of samples	Range	Mean	Standard deviation	Coefficient of variation (%)
Moisture content (%)	A - PC	91	12.9 - 82.9	32.8	10.3	31.4
	- BPC	116	16.1 - 89.1	43.6	13.3	30.5
	B	51	10.3 - 84.5	65.7	18	27.4
	C - PC	19	14.5 - 38	26.4	7.5	28.4
	- BPC	20	14 - 52	32	11.2	35
Initial void ratio	A - PC	35	0.62 - 1.7	0.91	0.25	27.5
	- BPC	53	0.58 - 2.1	1.22	0.32	26.2
	B	26	0.52 - 2.85	1.9	0.4	21.1
	C - PC	18	0.41 - 2.36	0.8	0.42	52.5
	- BPC	16	0.33 - 1.36	0.8	0.27	33.8
Bulk density (Mg/m ³)	A - PC	88	1.416 - 2.2	1.895	0.145	7.7
	- BPC	112	1.405 - 2.052	1.796	0.12	6.7
	B	39	1.424 - 2.052	1.577	0.153	9.7
	C - PC	18	1.48 - 2.16	1.934	0.151	7.8
	- BPC	16	1.69 - 2.24	1.93	0.137	7.1
Liquid limit (%)	A - PC	80	26 - 74	53.4	9.8	18.4
	- BPC	101	26 - 86	55.6	11.3	20.3
	B	41	26 - 81	64.3	11.2	17.4
	C - PC	18	33 - 75	49.5	10.6	21.4
	- BPC	18	31 - 68	48.8	11	22.5
Plastic limit (%)	A - PC	80	14 - 39	24.1	5	20.7
	- BPC	101	12 - 43	24.9	5.2	20.9
	B	41	18 - 62	29.3	7.9	27
	C - PC	18	15 - 32	21.6	4	18.5
	- BPC	18	14 - 27	21.8	3	13.8
Liquidity index	A - PC	80	-1.07 - 1.46	0.21	0.42	200
	- BPC	101	-0.93 - 1.32	0.56	0.35	62.5
	B	41	-0.68 - 4.29	1.12	0.63	56.3
	C - PC	18	-0.15 - 1.42	0.44	0.35	79.5
	- BPC	18	0.18 - 1.57	0.61	0.33	54.1
Overconsolidation ratio	A - PC	28	1.8 - 18.8	5.8	4	69
	- BPC	39	1.2 - 5.9	2.8	1	35.7
Torvane shear strength (kPa)	A - PC	59	12 - 158	66.3	35.6	53.7
	- BPC	61	4 - 145	41.7	23.5	56.4
	B	121	0 - 35	13.8	4.9	35.6

A- New Hong Kong Airport; B- Penny's Bay (samples undivided); C- Southeast Kowloon; PC- Palaeo-desiccated crust; BPC- Below palaeo-desiccated crust.

Coefficient of variation = Standard deviation x 100 / Mean %.

- (1) The mean moisture content at all three sites are all appreciably lower than the overlying M1 unit which is in agreement with desiccation caused by sub-aerial exposure. In addition to this, both the HKA and SK sites show increase in mean moisture contents in samples the below the palaeo-desiccated crust. The lowest mean value found at the SK site is explained by the greater particle-size variability. The highest mean value is found at the PB site where the escaping gas is indicated by acoustic turbidity in the seismic profiles (Yim et al. 2002a).
- (2) Much lower initial void ratios than in the M1 unit have been found in all three study areas. This is accounted for by cementation of the soil particles through iron oxides produced by the development of acid-sulphate soils during sub-aerial exposure (Yim and Tovey 1995; Tovey and Yim 2002) and is supported by the enrichment in iron content found in a drill-hole from the HKA site (Yim and Li 1983). The increase in coefficient of variation may be explained by post-depositional bioturbation through plant and animal activities under a terrestrial environment.
- (3) The increase in mean bulk densities compared to the M1 unit is accounted for by an increase in particle size, consolidation and cementation by iron oxides. The highest mean value is found for the SK site, followed by HKA and PB sites. This is thought to reflect the development of the palaeo-desiccated crust, with the SK site the best developed and the PB site the least well developed.
- (4) Liquid limit, plastic limit and liquidity index are all found to be lower in comparison to the M1 unit. The pattern can be explained by a combination of post-depositional desiccation, bioturbation and cementation accompanied by groundwater fluctuations during sub-aerial exposure of the M2 unit. A fall in the palaeo-groundwater table during the sub-aerial exposure would increase the effective stress to cause consolidation. The highest mean values found for liquid limit, plastic limit and liquidity index at the PB site are consistent with the presence of gas causing the palaeo-desiccated crust to be less well developed. At the HKA and SK sites, the increase in mean liquidity index below the palaeo-desiccated crust is consistent with palaeo-groundwater table fluctuations having a smaller impact at depth during the sub-aerial exposure of the unit.
- (5) The mean overconsolidation ratio is higher than in the M1 unit at the HKA site because a palaeo-desiccated crust formed by sub-aerial exposure is present. The lower overconsolidation ratio found in samples below the crust is consistent with a lower degree of desiccation at depth.
- (6) There is an increase in mean torvane shear strength compared to the M1 unit both at the HKA and PB sites. At the HKA site, a decrease is also found below the desiccated crust. The same reason used for explaining the overconsolidation ratio is applicable.

Table 4 shows the correlation coefficients between selected engineering properties in the palaeo-desiccated crust part of the last interglacial M2 unit of the three study areas. With few exceptions the strength of the correlation is weaker in comparison to the M1 unit. The greater weakness in correlation is particularly evident in torvane shear strength, *in situ* vertical stress and maximum past pressure. This is accounted for mainly by post-depositional changes affecting the M2 unit causing greater variability in the engineering properties as compared to the M1 unit which has never been sub-aerially exposed. At the HKA site, the strongest positive correlation is found between moisture content and initial void ratio (0.99) while the strongest negative correlations are between the initial void ratio and bulk density (-0.98), and moisture content and bulk density (-0.96). At the PB site, the strongest positive correlation is found to exist between moisture content and initial void ratio (0.83) while the strongest negative correlation is between initial void ratio and bulk density (-0.74). Unlike the other two sites, plastic limit also shows the weakest correlation with bulk density, moisture content (although insignificant), initial void ratio and liquid limit in decreasing order again reflecting

the influence of gas on engineering properties. At the SK site, the strongest positive correlation is found between moisture content and initial void ratio (0.86) while the strongest negative correlations are between initial void ratio and bulk density (-0.92), and moisture content and bulk density (-0.85).

Table 4 Correlation coefficients between selected engineering properties in the palaeo-desiccated crust of the last interglacial M2 unit of the New Hong Kong International Airport and Southeast Kowloon sites. Statistically insignificant correlation coefficients are marked by asterisks.

	Locality	Moisture content	Initial void ratio	Bulk density	Liquid limit	Plastic limit	Torvane shear strength	<i>In situ</i> vertical stress	Maximum past pressure
Moisture content	A	1.00	0.99	-0.96	0.20*	0.04*	-0.26*	-0.21*	-0.42
	C	1.00	0.26*	-0.35*	0.73	0.63	-	-	-
Initial void ratio	A		1.00	-0.98	0.24*	-0.04*	-0.29	-0.19	-0.42
	C		1.00	-0.94	0.20*	0.16*	-	-	-
Bulk density	A			1.00	-0.20*	0.003*	0.27*	0.16	0.41
	C			1.00	-0.25*	-0.27*	-	-	-
Liquid limit	A				1.00	0.46	-0.35*	0.19*	-0.51
	C				1.00	0.88	-	-	-
Plastic limit	A					1.00	-0.11*	-0.19*	-0.29
	C					1.00	-	-	-
Torvane shear strength	A						1.00	-0.23*	0.18*
	C							-	-
<i>In situ</i> vertical stress	A							1.00	0.18
	C							1.00	-
Maximum past pressure	A								1.00
	C								1.00

A- New Hong Kong Airport (23 samples); C- Southeast Kowloon (17 samples).

Table 5 shows the correlation coefficients between selected engineering properties in the below palaeo-desiccated crust part of the last interglacial M2 unit of the three study areas. In general, correlation is much better between the engineering properties than in the palaeo-desiccated crust and resembles those found for the M1 unit with the exception of torvane shear strength, *in situ* vertical stress and maximum past pressure. More strongly positive correlations are observed between moisture content and initial void ratio, moisture content and liquid limit, initial void ratio and liquid limit, moisture content and plastic limit, and liquid limit and plastic limit in comparison to the M1 unit in all three study areas with rare exceptions. A more strongly negative correlation is also observed between moisture content and bulk density, and initial void ratio and bulk density in comparison to the M1 unit in all three study areas. The similarity between the M1 unit which has never been sub-aerially exposed and the M2 unit below the palaeo-desiccated crust reinforces the view that the M2 unit is 'marine' origin with engineering properties similar to the M1 unit.

Based on the above results, the difference between correlation coefficients shown in Tables 4 and 5 should reflect the influence of sub-aerial exposure on the last interglacial M2 unit. The most dramatic changes caused by sub-aerial exposure as indicated by the correlation coefficients is the weaker correlation between initial void ratio and plastic limit, bulk density and plastic limit, bulk density and liquid limit, and initial void ratio and liquid limit. This change is likely to be the result of a combination of desiccation, bioturbation and cementation.

Table 5 Correlation coefficients between selected engineering properties in the below palaeo-desiccated crust part of the last interglacial M2 unit of the three study areas unless stated in the text. Statistically insignificant correlation coefficients are marked by asterisks.

	Locality	Moisture content	Initial void ratio	Bulk density	Liquid limit	Plastic limit	Torvane shear strength	<i>In situ</i> vertical stress	Maximum past pressure
Moisture content	A	1.00	0.99	-0.98	0.85	0.86	0.01*	-0.34*	-0.45
	B		0.90	-0.80	0.92	0.43	-	-	-
	C		0.93	-0.91	0.72	0.70	-	-	-
Initial void ratio	A		1.00	-0.98	0.83	0.86	-0.01*	-0.34*	-0.48
	B			-0.83	0.87	0.29*	-	-	-
	C			-0.98	0.70	0.71	-	-	-
Bulk density	A			1.00	-0.80	-0.85	0.02*	0.36	0.50
	B				-0.82	-0.19*	-	-	-
	C				-0.71	-0.78	-	-	-
Liquid limit	A				1.00	0.81	0.36	-0.27*	-0.07*
	B					0.43	-	-	-
	C					0.89	-	-	-
Plastic limit	A					1.00	0.08	-0.13*	-0.29*
	B						-	-	-
	C						-	-	-
Torvane shear strength	A						1.00	-0.08*	0.50
	B							-	-
	C							-	-
<i>In situ</i> vertical stress	A							1.00	0.28*
	B								-
	C								-
Maximum past pressure	A								1.00
	B								
	C								

A- New Hong Kong Airport (32 samples); B- Penny's Bay (24 samples); C- Southeast Kowloon (15 samples).

Table 6 provides a summary of selected engineering properties of the second last interglacial M3 unit. Although the number of samples tested for engineering properties is relatively small and it is not possible to differentiate between samples from the palaeo-desiccated crust and below the crust at the PB and SK sites, there are differences in engineering properties compared to the M1 and M2 units. The characteristics include:

- (1) The lowest mean moisture content is found in this unit. At the HKA site, an increase in mean moisture content is also present below the palaeo-desiccated crust.
- (2) The lowest mean initial void ratio is found in this unit. At the HKA site, an increase in mean initial void ratio is also present below the palaeo-desiccated crust.
- (3) The mean bulk density in most cases is slightly higher than the M2 unit and distinctly higher than the M1 unit. As in the M2 unit at the HKA site, the mean bulk density decreases slightly in the palaeo-desiccated crust compared to below the paleo-desiccated crust.
- (4) The mean liquid limit, the mean plastic limit and the mean liquidity index are lower than in the M1 unit but comparable to the M2 unit. At the HKA site, slight increases in mean liquid limit, the mean plastic limit and the mean liquidity index are found below the desiccated crust.
- (5) The mean overconsolidation ratio at the HKA site increases in the palaeo-desiccated crust. This feature is similar to the M2 unit.
- (6) The mean torvane shear strength at the HKA site increases in the palaeo-desiccated crust. This feature is also similar to the M2 unit.

Table 6 Statistical summary of selected engineering properties of the second last interglacial M3 unit in the three study areas.

Property	Locality	No. of samples	Range	Mean	Standard deviation	Coefficient of variation (%)
Moisture content (%)	A - PC	24	17.3 - 60.6	32.6	8.2	25.2
	- BPC	28	23 - 72.8	41.4	11.3	27.3
	B	34	15.8 - 46.4	24.9	7.7	30.9
	C	8	12.5 - 42	29.1	8.6	29.6
Initial void ratio	A - PC	9	0.49 - 1.39	0.77	0.27	35.1
	- BPC	16	0.49 - 1.83	1.1	0.36	32.7
	B	23	0.42 - 0.95	0.68	0.15	22.1
	C	8	0.39 - 1.2	0.71	0.25	35.2
Bulk density (Mg/m ³)	A - PC	24	1.634 - 2.12	1.887	0.136	7.2
	- BPC	26	1.589 - 2.054	1.802	0.122	6.8
	B	25	1.739 - 2.182	1.927	0.104	5.4
	C	8	1.76 - 2.14	1.977	0.137	6.9
Liquid limit (%)	A - PC	17	27 - 65	51	8.3	16.3
	- BPC	21	31 - 65	52.4	11.1	21.2
	B	27	31 - 85	51.1	12.5	24.5
	C	7	39 - 90	57.6	16.8	29.2
Plastic limit (%)	A - PC	17	12 - 30	22.6	4.2	18.6
	- BPC	21	14 - 38	23.6	6	25.4
	B	27	14 - 38	24.7	4.9	19.8
	C	7	17 - 27	22.9	4	17.5
Liquidity index	A - PC	17	-0.11 - 0.89	0.3	0.21	70
	- BPC	21	-0.11 - 1.68	0.65	0.38	58.5
	B	27	-1.03 - 0.71	-0.0008	0.4	>100
	C	7	0.24 - 1.14	0.59	0.33	55.9
Overconsolidation ratio	A - PC	6	2.1 - 4.3	3.4	0.8	23.5
	- BPC	7	1.3 - 2.9	2.4	0.5	20.8
Torvane shear strength (kPa)	A - PC	9	36 - 113	69.3	21.4	30.9
	- BPC	10	20 - 80	46	18.9	41.1

A- New Hong Kong Airport; B- Penny's Bay (samples undivided); C- Southeast Kowloon (samples undivided); PC- Palaeo-desiccated crust; BPC- Below palaeo-desiccated crust.

Coefficient of variation = Standard deviation x 100 / Mean %.

GENERAL DISCUSSION

Based on the results obtained, the engineering properties of the M1 unit are found to be closely related to the depositional environment while the engineering properties of the M2 and M3 units are found to be related both to the depositional environment and the post-depositional environmental changes which had taken place. There is therefore a close relationship between Quaternary environmental changes and the engineering properties of the transported soils. If the total geological history approach of Fookes et al. (2000) is adopted, the muddy inner continental shelf model is applicable to the M1 unit while that for the M2 and M3 units would require a muddy inner continental shelf model as well as one or two terrestrial models to take into account the development of palaeosol(s) and the aging effect on the sub-aerially exposed marine deposits.

Also based on the results obtained, it can be seen that the geotechnical model proposed by Chandler (2000) for clay sediments in depositional basins requires further refinement when applied to the Quaternary period to take into account the role of eustatic sea-level changes. Clay units formed by multiple transgressions should be delineated into individual units formed by a single interglacial-glacial cycle. This is necessary for both the depositional and post-depositional environmental changes which had taken place. Based on the geological

model proposed for Hong Kong, up to five interglacial-glacial cycles can be identified within circa 0.5 million years.

The development of palaeosols formed by the sub-aerial exposure of marine deposits during glacial periods of low sea-level stands is relevant to geotechnical engineering because of the strong influence on engineering properties. Castleberry II and Prebaharan (1985) identified palaeo-desiccated crusts (referred to by these authors as clay crusts) to be a hazard to jack-up operations for oil and gas exploration on the Sunda Shelf. The identification of these palaeosols is necessary to avoid punch-through failures. In the coastal waters of Hong Kong, past experience in the construction of coastal reclamations shows that dredging work to remove the M2 unit is notoriously slow in areas with well-developed palaeo-desiccated crusts such as the HKA platform (M. Peart, personal communication). This is explained by the overconsolidated state of the crust which is caused by cementation by iron oxides. On the other hand, shrinkage cracks commonly present within the crusts was used advantageously for the dissipation of pore pressures during the construction of the platform for HKA (Dr G. W. Plant, personal communication).

Other applications of the geological model beneficial to geotechnical engineering include:

(1) Logging of transported soils in drill-holes

As far as the authors are aware, the geological model proposed provides the best explanation of what is present in the drill-holes selected for examination in the present study. The practice of referring to a deposit as alluvium based on colour change and the absence of marine shells and shell fragments which is still widespread in Hong Kong is incorrect. For pre-Holocene marine deposits, it is necessary to consider the role of post-depositional environmental changes in altering the original characteristics of the deposits including their engineering properties. The development of acid-sulphate soils during sub-aerial exposure due to the oxidation of the mineral pyrite present in marine deposits would lead to the formation of sulphuric acid causing the calcareous shells and shell fragments to be either partially or completely destroyed (Yim and Tovey; Tovey and Yim 2002). At the same time iron produced as a bi-product would result in the cementation of the soil particles causing an increase in overconsolidation ratio and torque shear strength.

(2) Ground-truthing of geophysical investigations

All geophysical investigations require reliable site investigation including drilling for ground-truthing. For example, Yim et al. (2002c) have provided recommendations on how the interpretation of seismic profiles can be improved upon in order to maximize the amount of information to be gained. At the PB site, the influence of escaping deep-seated gas was initially identified from the presence of acoustic turbidity in seismic profiles and the presence of gas has an important effect of the engineering properties. In a study of the PB site by Wong (1996), the application of the geological model together with magnetic susceptibility testing of continuous drill-holes was demonstrated to assist seismic interpretation even in areas severely affected by acoustic turbidity.

(3) Exploration and exploitation of offshore aggregates

In a study of the stratigraphy of Quaternary offshore aggregates in Hong Kong applying the geological model proposed (Yim 2001), three main types of deposits have been identified. Firstly, intertidal and near-shore subtidal deposits at the mouth of major streams formed by the reworking of pre-Holocene colluvial-alluvial fans. Secondly, alluvial deposits of last glacial and/or earlier glacial periods buried by Holocene and/or earlier interglacial marine deposits. Thirdly, last interglacial marine deposits reworked during the Holocene

transgression. Therefore eustatic sea-level changes dating back to circa 0.5 million years have played a major role in the formation of the offshore aggregates.

(4) Age of formation of relic landslide deposits

The geological model provides a time framework for the recognition of relic landslide deposits formed during circa 0.5 million years. By applying the model, Yim (2000) demonstrated that partially drowned and drowned landslide deposits formed during different glacial periods can be identified. The landslide deposits which are significantly larger in size in comparison to the modern counterparts were probably caused by the fall in base level leading to a rapid drawdown of the water table. Further research work in this area would be valuable for reconstructing the age and magnitude-frequency relationships of the landslides including those occurring above present day sea level through the application of land-sea correlation. This would help to provide a better understanding of polycyclic terrestrial landscapes which are notoriously difficult to date in spite of recent advances of Quaternary dating methods.

(5) Future coastal engineering

While the threat of landslides is a hindrance to developing land covered by hillslopes, the creation of land for development by coastal reclamations would face new challenges as new reclamation sites are selected. The geological model proposed with five interglacial-glacial cycles provides a stratigraphic framework for accounting the sequence of transported soils expected to be present within the territorial waters of Hong Kong.

CONCLUSIONS

A circa 0.5-million year geological model, based on the total geological history approach of Fookes et al. (2000) and with refinements on the geotechnical cycle of Chandler (2000), for geotechnical engineering applications in Hong Kong is proposed. The model extending back in geological time to cover the last five interglacial-glacial cycles of the Quaternary period is the result of eustatic sea-level changes. Based on the statistical analysis of engineering properties of the M1, M2 and M3 units from the three study areas, the engineering properties are found to be closely related to Quaternary environmental changes. Conversely, the engineering properties of transported soils are found to provide invaluable information of past earth history. Since the depositional environment and post-depositional environmental changes are both important in determining engineering properties, the traditional view on the consolidation of transported soils needs to be modified to take into account the factors associated with post-depositional environmental changes. The present study shows that soil development of sub-aerially exposed marine deposits of pre-Holocene age during periods of low sea-level stands are an important contributing factor to the variability in their engineering properties.

REFERENCES

- Berner, R. A. (1984). "Sedimentary pyrite formation: an update." *Geochimica et Cosmochimica Acta*, 48, 605-615.
- Castleberry II, J. P., and Prebahren, N. (1985). "Clay crusts of the Sunda Shelf – a hazard to jack-up operations." Proc., 8th Southeast Asian Geotechnical Conference, Kuala Lumpur, 1.40-1.48.
- Chandler, R. J. (2000). "Clay sediments in depositional basins: the geotechnical cycle." *Quaternary Journal of Engineering Geology and Hydrogeology*, 33, 7-39.
- Davis, A. M., Chan, L. S., Yeung, C. H., and Yim, W. W.-S. (1999). "Application of magnetic susceptibility testing to offshore sediments in South East Kowloon

- development feasibility study." *Urban Ground Engineering*, Clarke, B., Editor, Institution of Civil Engineers, Thomas Telford, London, 287-295.
- Fookes, P. G. (1991). "Quaternary engineering geology." *Quaternary Engineering Geology*, Forster, A., Culshaw, M. G., Cripps, J. C., Little, J. A., and Moon, C. F., Editors, Geological Society Special Publication No. 7, The Geological Society, London, 73-98.
- Fookes, P. G. (1997). "Geology for engineers: the geological model, prediction and performance." *Quarterly Journal of Engineering Geology and Hydrogeology*, 24, 249-265.
- Fookes, P. G., Baynes, F. J., and Hutchinson, J. N. (2000). Total geological history: a model approach to the anticipation, observation and understanding of site conditions. Proc., GeoEng 2000, Melbourne, 370-460.
- Fyfe, J. A., Shaw, R., Campbell, S. D. G., Lai, K. W., and Kirk, P. A. (2000). *The Quaternary Geology of Hong Kong*. Geotechnical Engineering Office, Civil Engineering Department, The Government of the Hong Kong Special Administrative Region.
- Langford, R. L., James, J. W. C., Shaw, R., Campbell, S. D. G., Kirk, P. A., and Sewell, R. J. (1995). *Geology of Lantau District*. Hong Kong Geological Survey Memoir No. 6, Geotechnical Engineering Office, Civil Engineering Department, The Government of the Hong Kong Special Administrative Region.
- Petit, J.R., Jouzel, J., Raynaud, D., Barkov, N.I., Barnola, J.-M., Basile, I., Chappellaz, J., Davis, M., Delaygue, G., Delmotte, M., Kotlyakov, V. M., Legrand, M., Lipenkov, V. Y., Lorius, C., Pepin, L., Ritz, C., Saltzman, E., and Stievenard, M. (1999). "Climate and atmospheric history of the past 420,000 years from the Vostok ice core, Antarctica." *Nature*, 399, 429-436.
- Premchitt, J., Rad, N. S., To, P., Shaw, R., and James, J. W. C. (1992). "A study of gas in marine sediments in Hong Kong." *Continental Shelf Research*, 12, 1251-1264.
- Sewell, R. J., Campbell, S. D. G., Fletcher, C. J. N., Lai, K. W., and Kirk, P. A. (2000). *The Pre-Quaternary Geology of Hong Kong*. Geotechnical Engineering Office, Civil Engineering Department, The Government of the Hong Kong Special Administrative Region.
- Skempton, A.W. (1944). "Notes on the compressibility of clay." *Quarterly Journal of the Geological Society of London*, 100, 119-135.
- Terzaghi, K. (1927). "Principles of final soil classification." *Public Roads*, 8, p. 41.
- Tovey, N. K., and Yim, W. W.-S. (2002). "Desiccation of late Quaternary inner shelf sediments: microfabric observations." *Quaternary International*, 92, 73-87.
- Williams, M., Dunkerley, D., De Deckker, P., Kershaw, P., and Chappell, J. (1998). *Quaternary Environments*. 2nd edition, Arnold, London.
- Wong, J. C. F. (1996). *Marine Seismic Interpretation of the Container Terminals 10 and 11 site*. B.Sc. thesis, Department of Earth Sciences, The University of Hong Kong, Hong Kong.
- Yim, W. W.-S. (1992). Evidence from stratigraphy, fossils and dating. Proc., The Logging and Interpretation of Transported Soils in Offshore Boreholes, Geological Society of Hong Kong, Hong Kong, 19-30.
- Yim, W. W.-S. (1994). "Offshore Quaternary sediments and their engineering significance in Hong Kong." *Engineering Geology*, 37, 31-50.
- Yim, W. W.-S. (2000). "Drowned and partially drowned landslide deposits in Hong Kong." Proc., Symposium on Slope Hazards and Their Prevention, The Jockey Club Research and Information Centre for Landslip Prevention and Land Development, Hong Kong, 78-83.

- Yim, W. W.-S. (2001). "Stratigraphy of Quaternary offshore sand and gravel deposits in the Hong Kong SAR, China." *Quaternary International*, 82, 101-116.
- Yim, W. W.-S., Chan, L. S., Hsieh, M., Philp, R. P., and Ridley Thomas, W. N. (2002a). "Carbon flux during the last interglacial cycle in the inner continental shelf of the South China Sea off Hong Kong." *Global and Planetary Change*, 33, 29-45.
- Yim, W. W.-S., and Choy, A. M. S. F. (2000). "Some engineering applications of Quaternary sea-level changes in Hong Kong." Proc., Conference on Engineering Geology HK 2000, The Institution of Mining and Metallurgy – Hong Kong Branch, 211-218.
- Yim, W.W.-S., Huang, G., and Chan, L. S. (2004). "Magnetic susceptibility study of late Quaternary inner continental shelf sediments in the Hong Kong SAR, China." *Quaternary International*, 117, 41-54.
- Yim, W. W.-S., and Li, Q.-Y. (1983). "Sea level changes and sea-floor surficial deposits off Chek Lap Kok." *Geology of Surficial Deposits of Hong Kong*, Yim, W. W.-S., and Burnett, A. D., Editors, Abstracts No. 1, Geological Society of Hong Kong, Hong Kong, 48-59.
- Yim, W. W.-S., and Li, J. (2000). "Diatom preservation in an inner continental shelf borehole from the South China Sea." *Journal of Asian Earth Sciences*, 18, 471-488.
- Yim, W. W.-S., Price, D. M., and Choy, A. M. S. F. (2002b). "Distribution of moisture contents and thermoluminescence ages in an inner shelf borehole from the new Hong Kong Airport site, China." *Quaternary International*, 92, 35-43.
- Yim, W. W.-S., Ridley Thomas, W. N., and Chan, L. S. (2002c). "Maximization of benefits in ground investigation for mass transportation projects involving major coastal reclamations in Hong Kong." Proc., 22nd Annual Seminar on Geotechnical Aspects of Mass Transportation Systems, Geotechnical Division, The Hong Kong Institution of Engineers, Hong Kong, 235-244.
- Yim, W. W.-S., and Tovey, N. K. (1995). "Desiccation of inner continental shelf sediments during Quaternary low sea-level stands." *Geoscientist*, 5/4, 34-35.
- Yim, W. W.-S., Yu, K.-F., and Catt, P. (1997). "Quaternary paleobiogeography of Hong Kong: evidence from marine borehole WB7." *The Changing Face of East Asia During the Tertiary and Quaternary*, Jablonski, N. G., Editor, Centre of Asian Studies, The University of Hong Kong, Hong Kong, 382-399.

ACKNOWLEDGEMENTS

This research work is partially supported by the Research Grants Council of the Hong Kong Special Administrative Region, China (Project No. HKU 7126/00P and Project No. HKU 7024/03P). The Civil Engineering Department of the Hong Kong Government of the Special Administrative Region and the Hong Kong Airport Authority are thanked for providing access to ground investigation data and drill-hole samples from the three study areas. We would particularly like to thank P. Y. K. Au, Dr L. S. Chan, Dr L. J. Endicott, D. C. Koutsoftas, M. D. Howat (deceased), Dr G. Huang, J.R. Mulgrew, Dr A. R. Pickles, N. Phillips, Dr G. W. Plant, Prof. W. N. Ridley Thomas, Dr N. K. Tovey, I. Turkington, and R. Wood for their assistance. This paper is a contribution to the International Geological Correlation Programme Project no. 464 'Continental shelves during the last glacial cycle: knowledge and applications' and to the Commission on Coastal and Marine Processes of the International Union for Quaternary Research.

RECENT ADVANCE IN LANDSLIP WARNING SYSTEM

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Abstract: Hong Kong is a city with rugged topography. Landslides generally occur during periods of heavy rainfall. The Geotechnical Engineering Office (GEO) of the Civil Engineering Department and the Hong Kong Observatory (HKO) of the Government of Hong Kong Special Administrative Region operate a Landslip Warning System to alert the public to landslide hazard. By using information on a combination of real-time measured rainfall and rainfall forecast, and based on our understanding of the rainfall-landslide relationship, the Hong Kong Government is able to issue Landslip Warnings as appropriate. One purpose of the Landslip Warning is to alert the public to reduce their exposure to possible danger from landslides. The GEO and the HKO manage a raingauge system of over 100 automatic raingauges distributed over Hong Kong that measures rainfall intensities at 5-minute intervals. The GEO keeps comprehensive rainfall and landslide data since 1984, which facilitates the study of correlations between rainfall and landslides. This paper outlines the key components of the GEO Landslip Warning System and presents the algorithm for predicting the number of landslides by the established correlations between landslide frequencies and maximum 24-hour rainfall using simplified GIS approach. The incorporation of short-range rainfall forecast made by the HKO into the Warning System is also described.

INTRODUCTION

Hong Kong has a long history of landslides, the majority of which were triggered by rainfall. The Geotechnical Engineering Office (GEO) of the Civil Engineering Department (CED) and the Hong Kong Observatory (HKO) of the Government of Hong Kong Special Administrative Region operate a Landslip Warning System. Landslip Warnings are issued during times of heavy rainfall when it is predicted that numerous landslides will occur. The purposes of Landslip Warning are to alert the public to reduce their exposure to possible danger from landslides and to trigger the operation of an emergency system within government departments that mobilises staff and resources to deal with landslide incidents.

The operation of the Landslip Warning System requires knowledge of the real-time and forecast of rainfall intensities, and a good understanding of the relationship between rainfall and landslides.

THE GEO RAINGAUGE SYSTEM

The GEO and the HKO operate an extensive network of automatic raingauges, providing real-time rainfall data to the Landslip Warning System. The locations of the raingauges are shown in Figure 1. This network, which dates back to 1984, was upgraded in 1999 to improve coverage in the new development areas. The network now comprises 86 raingauges located throughout Hong Kong. The data capture, control and processing system,

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which was also upgraded in 1999, receives data from the 86 GEO field raingauge stations and an additional 24 automatic raingauges operated by the HKO.

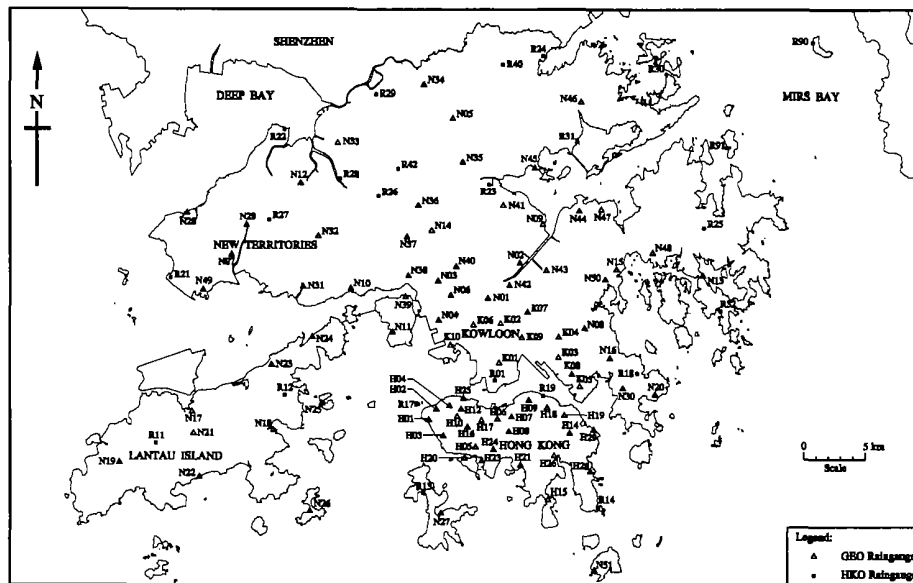


Figure 1 - Locations of GEO and HKO Field Raingauge Stations

The hardware for the GEO raingauge system includes two major parts: (i) 86 field raingauge stations and (ii) a Central Control Centre located in the Civil Engineering Department of the Government of Hong Kong Special Administrative Region. Each field raingauge station comprises a Casella tipping bucket raingauge, a data logger module with a modem for data transmission, and a rechargeable battery powered by a solar panel, as depicted in Figure 2. The Casella tipping bucket raingauge collects rainfall, and the data



Figure 2 - GEO Field Raingauge Station

logger module records the corresponding measurement in real-time. Every 5-minute the modem transmits the total rainfall amount recorded in the last 5-minute by private telephone line to the data acquisition unit in the Central Control Centre, which is equipped with

computers for receiving and processing the raw rainfall data (Figure 3). The data are also transmitted to the HKO in real-time using private telephone line.

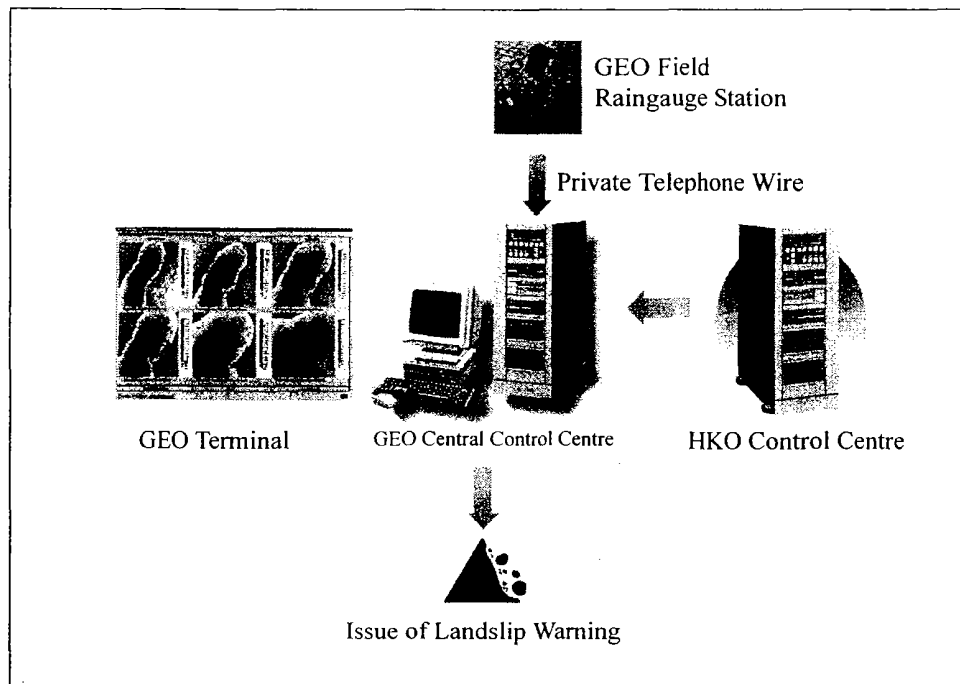


Figure 3 - GEO Landslip Warning System

Computer programs are developed and applied together with the proprietary software to acquire real-time rainfall data, display situation of rainfall development (such as, rainfall contour maps), calculate the predicted number of landslides and check automatically the rainfall situation against the Landslip Warning criteria. As mentioned in the above section, the real-time rainfall records are transmitted to the Central Control Centre every 5-minute. The software system then checks the validity of the rainfall data, updates the corresponding rainfall data and displays them in contour maps for different duration of rainfall (Figure 3). The Landslip Warning criteria are also automatically checked using the recorded and forecast rainfall and the rainfall-landslide correlation model. All the above processes are completed within five minutes.

CORRELATION BETWEEN RAINFALL AND LANDSLIDES

The first set of Landslip Warning criteria in 1977 was based on the work of Lumb (1975). Under the criteria, Landslip warnings would be issued when the 1-day rainfall exceeded a certain threshold value, which depended on the 15-day rainfall.

Brand et al (1984) reported on the work carried out in the early 1980s for the development of the second-generation Landslip Warning criteria. In that research, rainfall data covering a period of about 20 years were analysed. Data on the time of landslide occurrence used were based on the records of the Fire Services Department. Two thresholds values for the on-set of widespread landslides associated with 24-hour rainfall and one-hour rainfall respectively were established. The findings formed the basis of the Landslip Warning criteria in the second half of the 1980s and most of the 1990s.

Using the data on rainfall and landslides collected in the period 1984 to 1996 through the extensive raingauge network and detailed recording of landslide incidents by GEO's professional staff, the GEO carried out a comprehensive review of the Landslip Warning criteria in 1997 and 1998. Areas with relatively high landslide density (called 'vulnerable' areas) were identified and a correlation between landslide density and rolling 24-hour rainfall was established for these areas (Pun et al, 2003). Landslide density was defined as the number of landslides reported per unit area. The correlation model was adopted for estimating the number of landslides in real time and formed the basis of the third-generation Landslip Warning criteria till 2003. Landslip Warning would be issued when the estimated number of landslides exceeded a threshold value, currently set at 15. This value was adopted because past statistics showed that on average there was one major landslide (defined as a landslide with failure volume greater than 50 m³) in every 15 landslides.

Pang et al. (2000) reported using a simplified GIS approach to obtain a good correlation between slope failure rate, maximum rolling 24-hour rainfall and slope characteristics for soil cut slopes. Based on this, a new model for the rainfall-landslide correlation has been developed. In the model, Hong Kong is divided into about 700 spatial grid cells, each having a plan area of 1.5 km by 1.2 km. This follows the gridding system used for 1:1000 scale maps published by Lands Department of the Hong Kong Government. In total, 118 rainstorm events with a maximum rolling 24-hour rainfall exceeding 50 mm recorded at the HKO were identified for the period 1992 to 2001. Isohyets of maximum rolling 24-hour rainfall was prepared for all the rainstorms. From the maps, the maximum 24-hour rainfall values for each grid cell during each rainstorm was determined. An example is given in Figure 4 for the rainstorm on 1-4 July 1997.

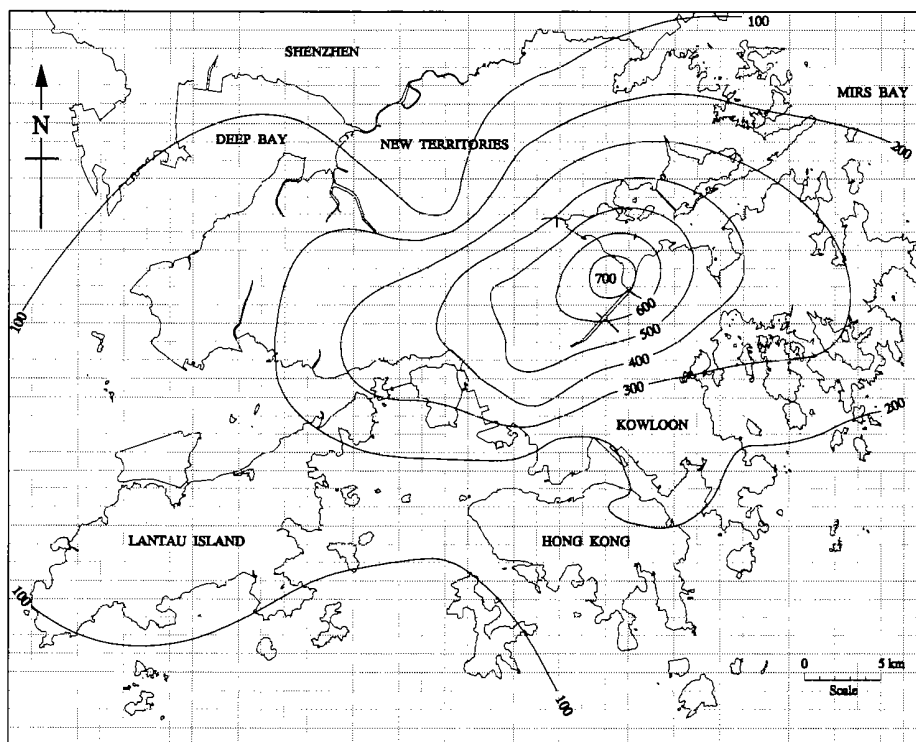


Figure 4 - Rainstorm 1-4 July 1997 (Maximum 24-hour Rainfall)

A total of 49040 man-made slope features (hereinafter referred to as 'slopes'), based on the digital data available in the GEO Slope Information System (SIS) (Lam et al, 1998), as of July 2001, were used for the study. The distribution of slopes and landslides is shown in Figure 5.

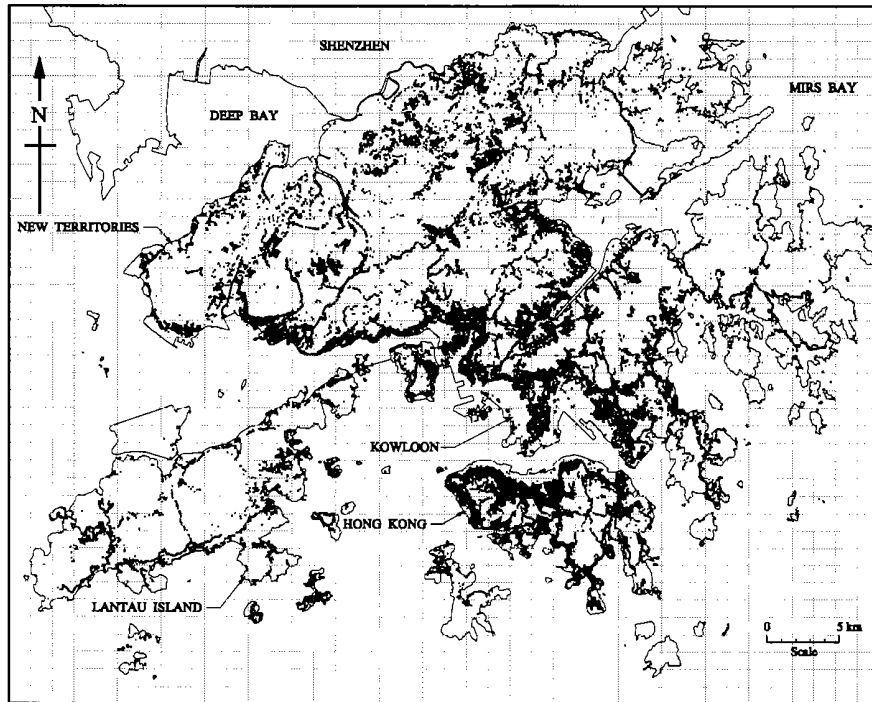


Figure 5a - Man-made Slope Features in Hong Kong (49040 Nos.)

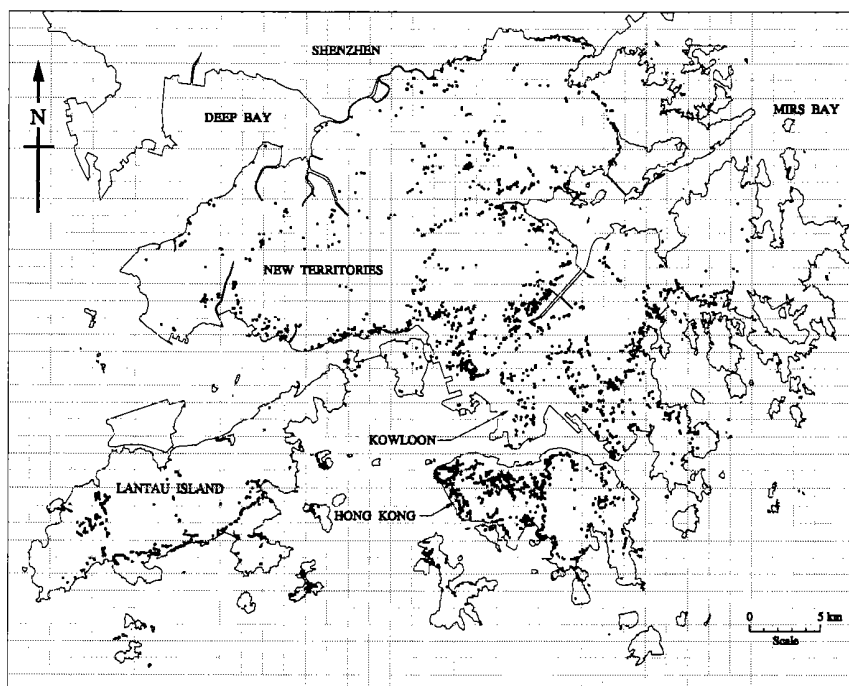
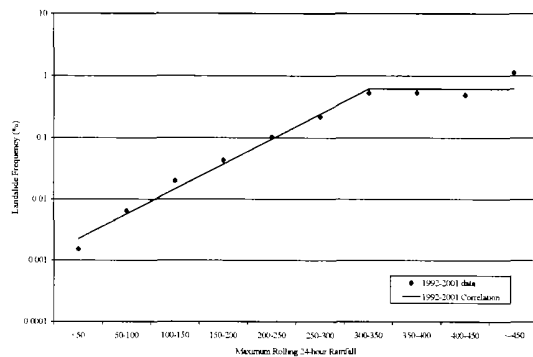
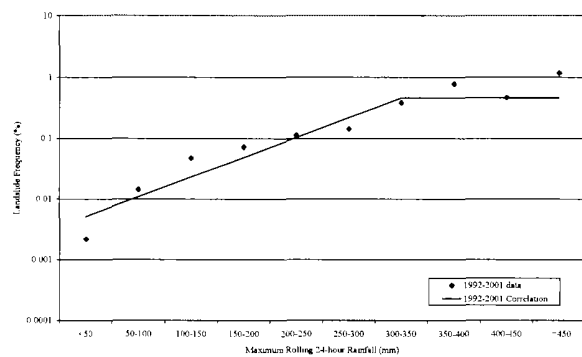


Figure 5b - Genuine Landslides at Man-made Slope Features within the Period 1992 - 2001 (3765 Nos.)

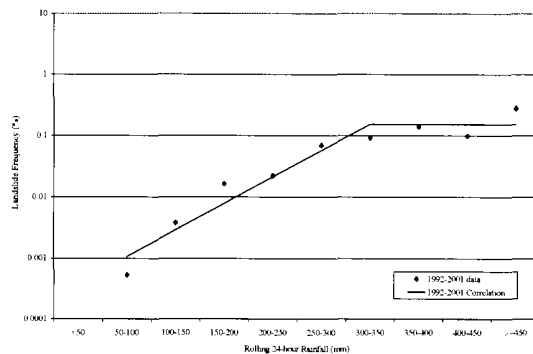
For each rainstorm, the number of landslides reported to the GEO was counted for each grid cell. As the total number of slopes in each grid cell was known, the landslide frequency could be calculated. A relationship between the amount of maximum rolling 24-hour rainfall and landslide frequency was then established by regression analysis. In light of the uncertainty in rainfall contouring, the regression analysis was performed on rainfall classes instead of the exact amount of rainfall. The rainfall classes used are 0-50 mm, > 50-100 mm, > 100-150 mm, > 150-200 mm, > 200-250 mm, > 250-300 mm, > 300-350 mm, > 350-400 mm, > 400-450 mm and ≥ 450 mm. Figure 6 shows the correlation models for different slope types. The landslide frequencies plotted in the figure were average values for all grid cells and all the rainstorms corresponding to each rainfall class.



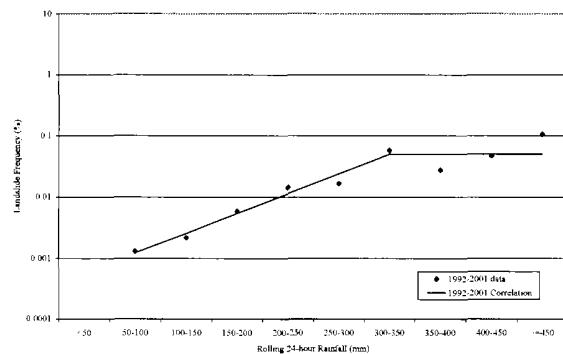
(a) Soil Cut Slopes



(b) Rock Cut Slopes



(c) Fill Slopes



(d) Retaining Walls

Figure 6 - Correlation between Landslide Frequency and Rolling 24-hour Rainfall

The advantages of the new correlation model over the previous ones are that:

- It takes into account the number and types of slopes in each 1.2 km x 1.5 km grid cell.
- The rainfall-landslide correlation for different types of slopes can be easily updated using new data.
- It can easily be extended beyond areas of correlation by adding grid cells, with knowledge of the number and types of slopes within the grid cells.

The performance of the new correlation model was evaluated for rainstorms in the period 1997 to 2001. The results were found satisfactory. Figure 7 shows the estimated number of landslides in the rainstorms against the actual number.

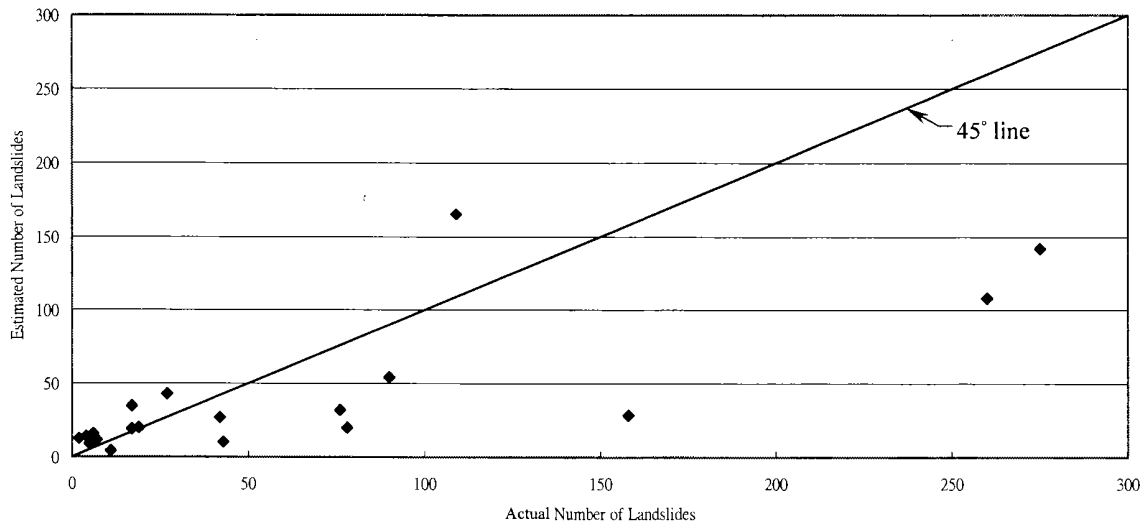


Figure 7 - Estimated and Actual Number of Landslides for Rainstorms Occurred between 1997 and 2001

LANDSLIP WARNING

The new rainfall-landslide correlation has been adopted since April 2004 for predicting the number of landslides in the existing Landslip Warning System. As the predicted landslide frequencies are averaged frequencies for the different slope types over Hong Kong, the correlations are not specific to any area. When there is a rainstorm, the 24-hour rainfall is calculated for all spatial grid cells. The predicted number of landslides in each grid cell is obtained by multiplying the landslide frequency determined by the past 24-hour rainfall and the number of slopes within the grid cell. The total number of predicted landslides in Hong Kong is the sum of predicted landslides of all the spatial grid cells. Figure 8 shows the distribution of predicted number of landslides for the rainstorm on 1-4 July 1997.

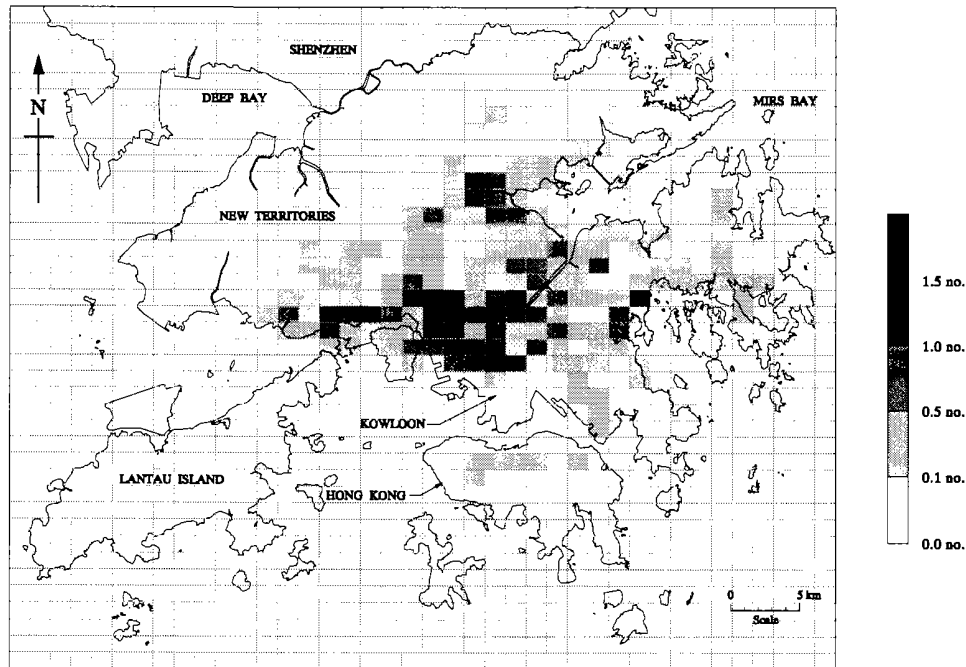


Figure 8 - Predicted Number of Landslides for the Rainstorm on 1-4 July 1997

Every 5 minutes, the GEO Landslip Warning System automatically calculates the total predicted number of landslides using the 24-hour rainfall data. As the use of the GEO Landslip Warning System is to issue timely warnings to the public and to mobilize Government staff and resources to deal with landslide incidents, Landslip Warning should be issued when the Landslip Warning Level (i.e. when the estimated number of landslide is 15 or more) is expected to be reached. Rainfall thus need to be forecasted. Under the current system, the estimated total number of landslide is made up of two components (i) landslide calculated for the past 21-hour and (ii) landslide calculated for the next 3-hour rainfall forecast. With this approach, a 3-hour lead-time can be allowed for emergency preparedness.

SHORT-RANGE RAINFALL FORECAST

Rainfall forecast is provided by the HKO. A recent technical development is called the Short-range Warning of Intense Rainstorms in Localized Systems (SWIRLS) by the use of weather radars (Li et al. 2000). This system relies on real-time correlation between radar echo intensity and rainfall amount recorded on the ground by raingauges and prediction of rain cloud movement. As indicated by Lam and Lam (2003), the system has the capability to provide reliable guidance to forecasters for making decision in respect to the issue of Landslip Warning. The rainfall forecast given by SWIRLS has been incorporated into the GEO Raingauge System to provide reference information for deciding the issue of Landslip Warning.

CONCLUSIONS

The methodology for predicting number of landslides by the established correlations between landslide frequencies and maximum 24-hour rainfall using a simplified GIS approach is given. The incorporation of the short-range rainfall forecast using SWIRLS developed by HKO into the Warning System is also discussed. On average, there are about three hundreds landslides reported to GEO and 3 to 4 Landslip Warnings issued each year. Continuous improvements to the GEO Landslip Warning System are essential for timely issuance of reliable warnings to the public.

REFERENCES

- Brand E.W., Premchitt, J. and Phillipson, H.B. (1984). Relationship between rainfall and landslides in Hong Kong. Proceedings of the 4th International Symposium on Landslides, Toronto, Vol. 1, pp 377-384.
- Lam C.C.L., Mak, S.H. & Wong, A.H.T. (1998). A new slope catalogue for Hong Kong. Proceedings of the Annual Seminar on Slope Engineering in Hong Kong, Hong Kong. A.A. Balkema Publishers, Rotterdam, pp. 235-242.
- Lam C.Y. and Lam C.C. (2003). From radar echoes to Landslip Warnings. Proceeding of Intelligent Engineering Applications of Digital Remote Sensing Technology, The Hong Kong Institution of Engineers, pp 35-41.
- Li P. W., Wong W.K., Chan K.Y. and Lai E.S.T. (2000). SWIRLS – An Evolving Nowcasting System. Technical Note, No. 100, Hong Kong Observatory 28p.
- Lumb P. (1975). Slope failures in Hong Kong. Quarterly Journal of Engineering Geology, Vol. 8, pp 21-65.
- Pang P. L. R., Pun W. K. and Yu Y. F. (2000). Estimation of failure frequency of soil cut slopes using rainfall and slope information. GeoEng2000, Australia.
- Pun W. K., Wong A. C. W. and Pang P. L. R. (2003). A review of the relationship between rainfall and landslides in Hong Kong. Proceedings of the 14th Southeast Asian Geotechnical Conference, Vol. 3, pp 211-216.

ACKNOWLEDGEMENT

This paper is published with the permission of the Head of the Geotechnical Engineering Office and the Director of Civil Engineering, Government of the Hong Kong Special Administrative Region.

SUSTAINABILITY CONSIDERATIONS IN THE DESIGN OF SLOPE WORKS

K. P. Yim¹

Abstract: Slope works is one of the major construction activities in Hong Kong, consuming a lot of natural resources and energy, and has major impacts on the built environment. At present, the industry's emphasis is mainly on visual impact. A great deal of other important aspects are in need of attention, such as, energy efficiency, the 3Rs approach in C & D materials, life cycle assessment, protection of wildlife habitats, ecological enhancement, and built heritage conservation. These considerations are likely to lead us to choose new materials, and to develop new technologies, new principles of slope treatment methods, and new systems. On the other hand, slope works is also a costly construction activity. We should try every effort to increase the return of the works, be it tangible or intangible. This is particularly so, as we are dealing with less risky slopes today after over two decades of very intensive treatment of Hong Kong slopes. As such, we should promote the generation of innovative ideas to turn slopes into asset, in addition to maintaining stability of slopes. Examples are slope advertisement, solar energy panels, grass skiing, ECO trails, rock carving, slope farming, etc. Many of the ideas can generate revenue and employment, create recreational values, improve environment, promote technology and boost tourism. This paper attempts to discuss how sustainability of slope works is viewed and how it impacts the built environment. It also looks at issues surrounding and enhancing the sustainable values of slope works, which could be implemented in the planning, design, construction and maintenance of slopes. Wherever applicable, the principles are also illustrated with case histories.

INTRODUCTION

Over the last few decades, Hong Kong has achieved tremendous success in overcoming the landslide problems and meeting the community's needs for the highest standards of slope safety, partly through a very substantial amount of slope works. Today, public opinion has been shifting towards the environment and the legacy that we will pass on to future generations. Society is beginning to realize that it must redirect its course towards a sustainable future. Realizing that slope works helps to make slopes safer, the community has also been continuously pressing for a more sustainable approach to be adopted in slope construction activities. The main concerns are alleviation of environmental impacts brought about by slope works and enhancement of the appearance of the engineered slopes. The engineering profession is responsible for the design and construction of slope works and should therefore take the leading role in promoting and practicing the principles of sustainable development in slope works. With the objective to achieve a healthier living environment and better quality of life for the community at large, the profession, the construction sector and the slope owners should properly manage the environmental issues arising from the design, construction and maintenance of the man-made slopes. The major challenge is to strike a good balance among the social, economic and environmental components of sustainability. This paper presents an understanding of the issue

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and a set of considerations and criteria on sustainable slope works, which have been implemented in the planning, design, construction and maintenance of slopes in Hong Kong.

SUSTAINABLE SLOPE WORKS

The most common definition of sustainable development is that presented in the 1987 World Commission on Environment and Development (1987), often known as the Brundtland Commission. The Commission defined Sustainable Development as development that meets the needs of the present without compromising the ability of future generations to meet their own needs. The concept of sustainable development has also been defined by the SUSDEV21 Study prepared by the Planning Department as a first step in the promotion of sustainability and sustainable development in Hong Kong:

“Sustainable Development in Hong Kong balances social, economic, environmental and resource needs, both for the present and future generations, simultaneously achieving a vibrant economy, social progress and a high quality environment, locally, nationally and internationally, through the efforts of the community and the Government.”

It covers areas, including economy, health and hygiene, natural resources, society and social infrastructure, bio-diversity, leisure and cultural vibrancy, environmental quality and mobility. Most of these areas are of direct relevance to engineering works, and need to be considered by the engineering profession in sustainable construction.

Sustainable engineering design and construction, including slope works, is a global trend. It is about strengthening and wherever relevant introducing environmental, socio-cultural and economic values into the practices of building and construction, including slope works, in a balanced act and with a view to achieving sustainable development. It is important not to mix with the concepts of “sustainable construction” and “green construction”. The concept of “green construction”, where only environmental issues (i.e. the environmental impacts of building and construction) are considered, is much narrower than the concept of “sustainable construction”. Sustainability goes wider than this to include, for example, contribution to economic growth and employment, cultural heritage, health and safety, etc.

Essentially, sustainable slope works is about minimizing the negative impacts and maximizing the positive impacts to achieve a balance in terms of environmental, economic and social performance. Some of the specific areas of concern are value management, life cycle assessment, more energy efficient designs, abatement of environmental nuisance during construction, reduction and management of construction and demolition materials, built heritage conservation, etc. The profession can help to ensure that these issues are considered throughout the planning and design process that sustainable option, sustainable materials and construction methods are chosen, and that high environmental standards pertain in the built facilities. A set of sustainability considerations pertinent to sustainable slope works design is identified as below: -

- Value for money
- Blending works with surroundings
- Minimization of disturbance to natural terrain
- Energy efficient design
- Reduction and utilization of waste materials
- Protection of wildlife habitats
- Built heritage conservation
- Increasing rate of return
- Life cycle assessment

The above criteria have been adopted in the design of local civil engineering and building projects, which have been under construction or completed recently in Hong Kong. The sustainability aspects of slope works design and construction are illustrated with examples as follows.

VALUE FOR MONEY

Value Management (VM) or Value Engineering technique has been successfully employed and has been widely regarded as a very useful tool these days to help finding **value-for-money** solutions for development projects. To meet the need for value-for-money, applying VM technique has become a pre-requisite in almost every project, also applicable to slope works.

VM could be defined as an organised methodology applied to the analysis of functions, components, goods and services, from the point of view of the system as a whole, to satisfy the required functions of the project at the lowest total cost without compromising quality and standard of performance. Typically, the VM process involves the holding of VM workshops conducted by experienced VM facilitator, as early as possible in the life cycle of a project.

In view of the relatively small size of most of the slope works projects, it is often unnecessary to hold VM workshops. However, the project team can capture the essence of VM principles, and rely on the initiatives of the project team members. There is also no limitation on when in the life cycle of a project that VM should be carried out and there is no such thing as the best timing. What is more important is the creation of the culture and environment, in which the colleagues have the urge and courage to:

- (i) question assumptions, fundamentals, and the authorities,
- (ii) be innovative and creative,
- (iii) be ready to break bureaucracy, and to overcome hurdles,
- (iv) be prepared to face challenges and to break deadlock situation.

Experience shows that the benefit of applying VM technique is so large that we should have practiced more with it. Yim & Chan (2003) describes a case history of a highway footbridge improvement project, in which the VM principles were applied to seek for simple solution to a complex geotechnical engineering problem (Figure 1). By proposing a change in the layout of a pedestrian overpass landing structure (Figure 2), it was able to do away with a highly risky bored pile wall construction, yet achieving a very substantial saving of over 40 million Hong Kong dollars. The construction duration was also cut down by at least a whole year of time.

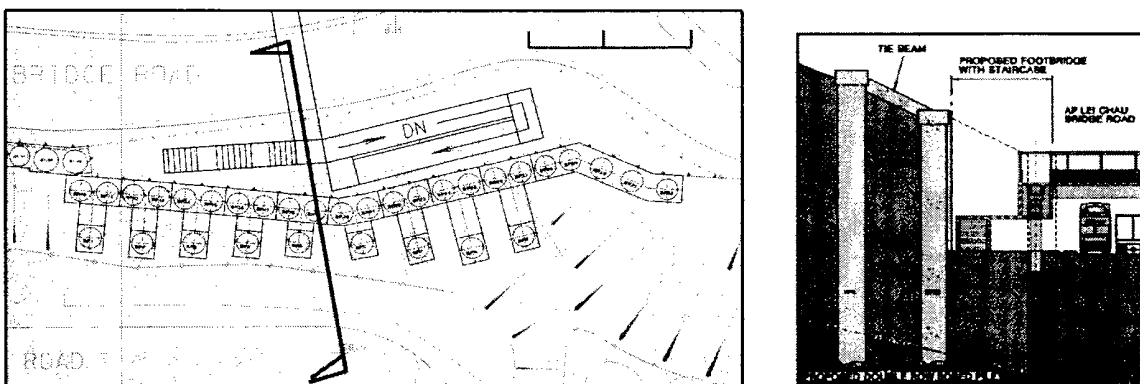


Figure 1. The original double row bored pile wall option (Yim & Chan (2003))

The case history shows that it is vital to maintain good communication and clear understanding of the client's basic functional requirements among project team members, the clients and stakeholders at all phases of the project. Reviewing and questioning basic assumptions at all phases of the project should be encouraged, although it is not always easy. The possible improvements and savings can sometimes be very large and even mean the difference between a feasible project and no project at all, as is demonstrated by this case history.

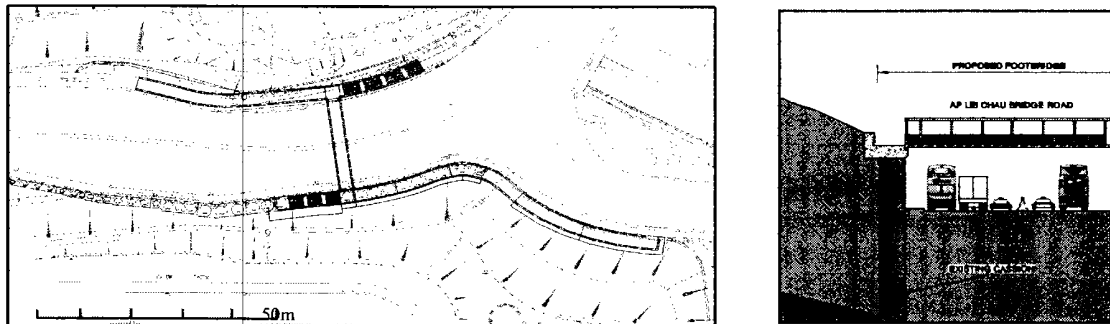


Figure 2. The revised scheme (Yim & Chan (2003))

VM could be defined as an organised methodology applied to the analysis of functions, components, goods and services, from the point of view of the system as a whole, to satisfy the required functions of the project at the lowest total cost without compromising quality and standard of performance. Typically, the VM process involves the holding of VM workshops conducted by experienced VM facilitator, as early as possible in the life cycle of a project.

BLENDING WORKS WITH SURROUNDINGS

Hong Kong has an extensive dense urban development located on or close to steep hillsides. Huge increases in slope construction over the last decade, coupled with the extensive use of shotcrete as a quick and durable means of slope surfacing, became a focus of community concern in the late 1990s. Public opinion has been a driving force for change in the approach to the local landscaping. The use of shotcrete on Government slopes now has to be justified on a case-by-case basis and is vetted by committees chaired by senior officials in all works departments.

There is broad scope in Hong Kong for application of alternative aesthetic and landscaping treatments in new site formation works and restoration of degraded natural terrain. Native vegetation species are under-represented in current hydro-seeding and transplanting practice. Local research is underway with a view to promoting greater use of native species on slopes, especially shrubs.

There are several techniques for improving the appearance of steep slopes. Wire netting rather than shotcrete has been provided on the slope face to prevent dislodgement of rock wedges. Netting tends to allow the natural colour and structure of the rock to be seen and it would also facilitate the establishment of creepers on the slope. As blending man-made slopes with the surrounding natural terrain is one of the criteria of sustainable slope works, the use of erosion control mat to facilitate the long-term enhancement of vegetation stability and also to reduce visual prominence of the man-made features, such as concealed soil nail heads on slope surface, has been successfully adopted in a number of large scale civil engineering projects and slope upgrading works. Several innovative techniques for establishing vegetation growth on steep slopes are being used. A pilot vegetation trial, which employs the technique of fiber-reinforced soil, has been implemented to assess the effectiveness of quick greening effect on steep rock cut slopes for a major highway project (Mak, 2003a).

Mak (2003b) described that the trend of slope surface protection is from the past Hard Landscape Approach (closed system basically composed of concrete facing), to a Soft Landscape Approach (Closed/Semi-closed system of green decorative cover on concrete surface), and recently to an Eco-Engineering Approach (open system simulating the natural environment as far as practicable). Descriptions of the three systems are given in Table 1.

<u>Hard Landscape Approach</u> Closed System	<u>Soft Landscape Approach</u> Closed / Semi-closed System	<u>Green(Eco-Engineering) Approach</u> Open System
Concrete Facing Slopes	Green Slopes Green Decorative Cover on concrete surface	Environmental Slopes
Shotcrete Rubble Pitch	• Erosion Mats with Hydroseeding / Hydromulching • Modified Multi-Layer Mats & Mulching • Turf Pots / Turf Tiles / Turf Gabion • Long Rooted Grass • Cemented Soil Fiber Mulch	Continuous Fiber Reinforced Soil (CFRS)
◆ Impermeable ◆ Prevent infiltration & erosion • No Vegetation • Facilitate Subsoil Drain with weep holes • Periodic Checks & Maintenance for integrity of membrane and underlying subsoil conditions ◆ Before 2000	◆ Impermeable / Semi-impermeable • Limit infiltration & prevent erosion ◆ Hide unpleasant bare surface ◆ Maintenance can be more costly - need to address vegetation cover, stability of the support elements for the vegetation and underlying concrete membrane integrity. ◆ Ornamental vegetation cover not sustainable - require maintenance to keep 'green' • Under vegetation cover -period checks & maintenance for integrity of membrane and underlying subsoil conditions ◆ Green but not environmental nor natural. • Since 2000	◆ Simulate Natural Slope ◆ Permeable and prevent erosion • Sustainable Vegetation Growth to promote natural succession ◆ Re-establishment of Ecological Equilibrium ◆ Maintenance is minimal when restored to natural habitat. ◆ Since 2001

Table 1. Slope protection systems (Mak, 2003b)

The Eco-engineering approach requires firstly the knowledge of the ecosystems within the surrounding areas likely to enable the process of natural invasion and hence the restoration of the ecosystems in the area concerned. Secondly, it needs the provision of a stable environmental skin using continuous fiber reinforced soil method (Geofiber) to ensure the smooth transition of vegetation on the slope surface to take place. Of particular interest is the environmental skin needs to promote root development and at the same time, it has to be strong enough to resist the severe bombardment of the severe Hong Kong weather. With increasing conscience on our health and our environment, we are aspired with the beauty of nature and seek to understand more on this natural environmental force that provides us the living world. The proposed

Eco-engineering approach serves to provide a new dimension on slope management in Hong Kong. Concerted efforts from all parties are necessary to enable valuable information be shared. We are now in the era of Eco-approach to engineer solutions, and it is timely that we can treat our slopes in Hong Kong in an Eco-manner.

Earth retaining structures, particular those constructed on hilly terrain to support building platforms and highways generally have a major visual impact on the surrounding. Conventional concrete retaining walls are visually intrusive and treatments to enhance their appearance are normally expensive. Reinforced fill retaining structures, which are constructed with prefabricated elements, allow greater flexibility in integrating aesthetic elements to achieve innovative and practical aesthetic design solution. Recently, a high reinforced fill wall constructed on hilly terrain to support a major highway is enhanced by a random combination of smooth and ribbed face pre-cast concrete panels, as illustrated in Figure 3. As the appearance of the concrete panels will deteriorate with time, planters are provided along the wall toe to facilitate the growth of local creeper species on the wall face to reduce the visual prominence of the wall.

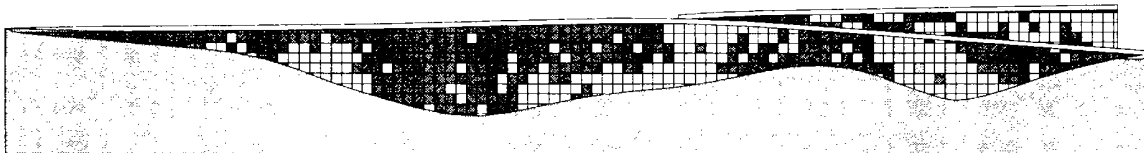
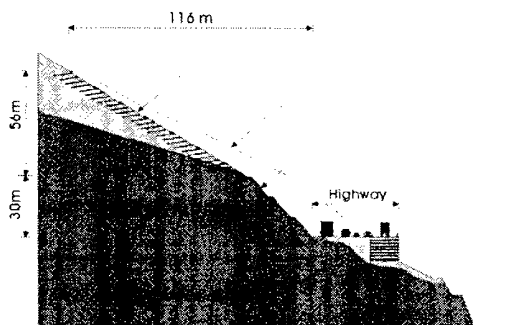


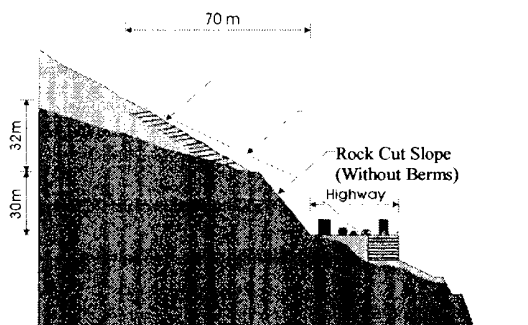
Figure 3. Architectural wall pattern of the reinforced fill wall (Lam et al 2001)

MINIMIZATION OF DISTURBANCE TO NATURAL TERRAIN

One of the prime objectives of sustainable slope works is to minimize disturbance to the natural terrain. The omission of berms on high rock cut slope, as illustrated in Figure 4, could drastically reduce land-take and excavation, hence, minimizing the disturbance to the natural terrain.



(a) 30m high rock cut slope with berms



(b) Omission of rock berms reduces excavation, slope stabilization works and disturbance to natural hillside

Figure 4. The effects of rock berms on cut slope design (Lam & Yim 2001)

When constructed on hilly terrain, reinforced fill embankments can be built with much steeper side slopes than conventional fill slopes, hence, minimizing disturbance to the natural hillsides and visual impact of large fill slopes. Natural terrain hazard mitigation measures (i.e. protective barriers against natural terrain landslides and boulder falls) can be provided within the lowland areas well away from the hinterland region. This arrangement would facilitate long-term maintenance works (i.e. clearance of debris) and also minimize the disturbance to the

natural terrain.

Eco-engineering approach serves to provide a new dimension on slope management in Hong Kong. Concerted efforts from all parties are necessary to enable valuable information be shared. We are now in the era of Eco-approach to engineer solutions, and it is timely that we can treat our slopes in Hong Kong in an Eco-manner.

ENERGY EFFICIENT DESIGN

Energy is one of the ecological parameters needed to determine the overall effects (short-term, long-term and associated) of engineering works. Of growing importance are the problems created by scarcity of raw materials, the environmental problems created by pollution, both of the atmosphere as well as the land from mining activities, the increase in manpower and transportation costs and the cost of maintenance. The choice of materials and structural forms used for any slope or retaining wall design influences all these parameters. Determination of the complete costs to the community of slope works may be attempted by studying the ecological parameters represented in the whole life cycle necessary for its production. However, a simplified approach is to compare the energy efficiency of different slope/retaining wall designs by studying their embodied energy (i.e. energy employed in producing and transporting materials and the energy consumed for construction, maintenance and demolition).

Adoption of the reinforced fill technique reduces the energy used in the construction of slopes and earth retaining structures (Figure 5). High reinforced fill (RF) wall is particularly energy efficient when compared with a conventional reinforced concrete (RC) wall (Lam & Yim, 2001). This is because the quantity of the structural elements (i.e. facing and reinforcement) forming a RF wall is significantly less than that required for a RC wall.

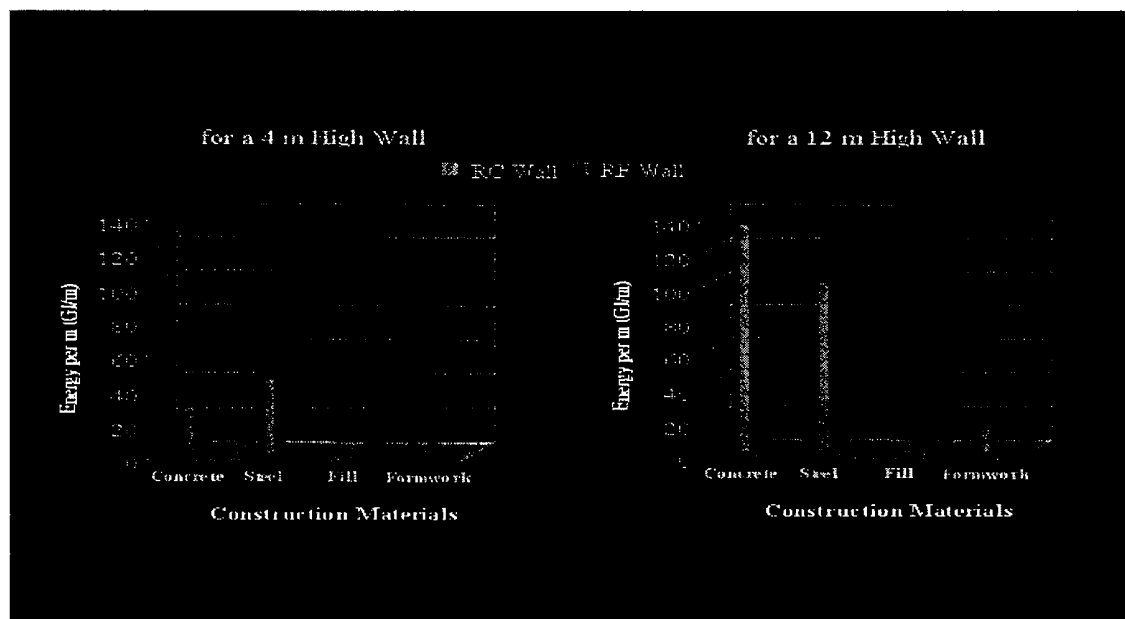


Figure 5. Energy comparison of reinforced fill (RF) wall and reinforced concrete (RC) wall

For soil cut slopes, conventional drainage design by providing surface channel on level-berms that have gentle gradients is vulnerable to blockage by debris and grass. Slopes with level-berms also need step-channels and down-pipes at intervals on the slope face to collect water from surface channels on berms. In order to reduce long-term maintenance of the slope drainage system, provision of sloping-berms along man-made slopes allows the drainage

channels to be self-cleansing, hence, minimizing long-term maintenance. In addition, the provision of visually intrusive step-channels and down-pipes on the slope face could be minimized. Sloping-berm design tends to fit with the natural topography better, and it usually results in the reduction of the total volume of excavation in comparison with the conventional level-berm design, hence, the reduction of energy required for the slope formation. An illustration of the sloping-berm arrangement is given in Figures 6 and 7.

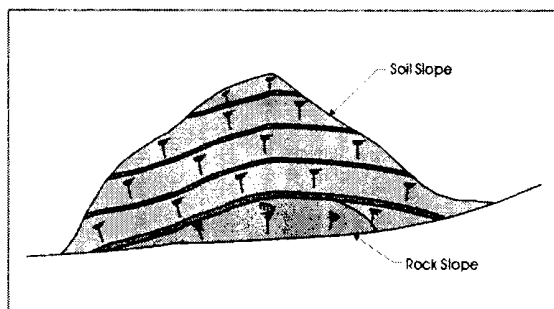


Figure 6. Sloping-berm

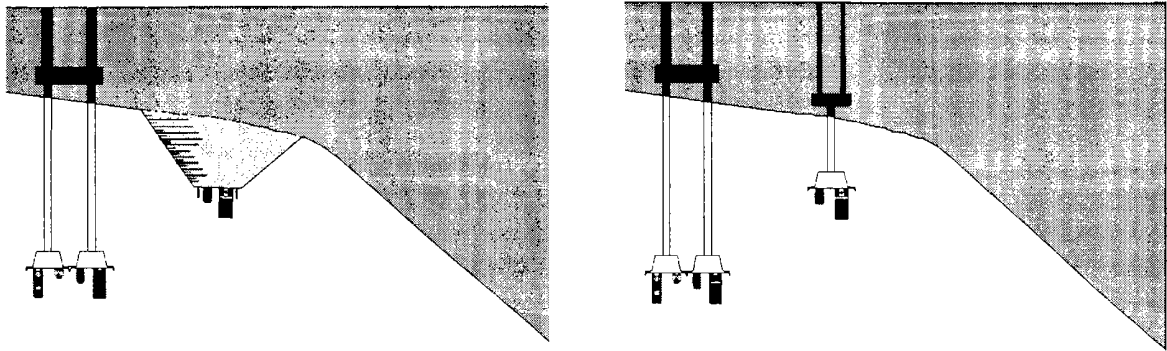
Figure 7. Sloping-berm cut slope (Lam & Yim 2001).

REDUCTION AND UTILIZATION OF WASTE MATERIALS

Hong Kong is currently facing with a great challenge in managing waste generated from its construction and demolition (C&D) activities. The scarce supply of land and the large-scale infrastructure works to cater for the rapid growth of population in the next 10 years have exacerbated the problem. Resolving the waste problem at source by avoiding or reducing waste generation is more effective and sustainable than resolving the waste problem after waste has been generated.

A task force on waste reduction for the construction industry was set up in 1999. The task force comprises representatives of professional bodies, academic institutions, policy bureau and works departments of the government as well as developers and contractor associations. Apart from government officials, there are also representatives from the private sectors. They work collaboratively with the government officials in the task force to propose and implement waste reduction measures. Recycled aggregates from C & D wastes have also been produced and used in concrete production, granular filling materials, road sub-base materials, paving blocks, rockfill replacement for seawall, gabion walls, fill slopes, etc. (Fong & Yeung, 2003).

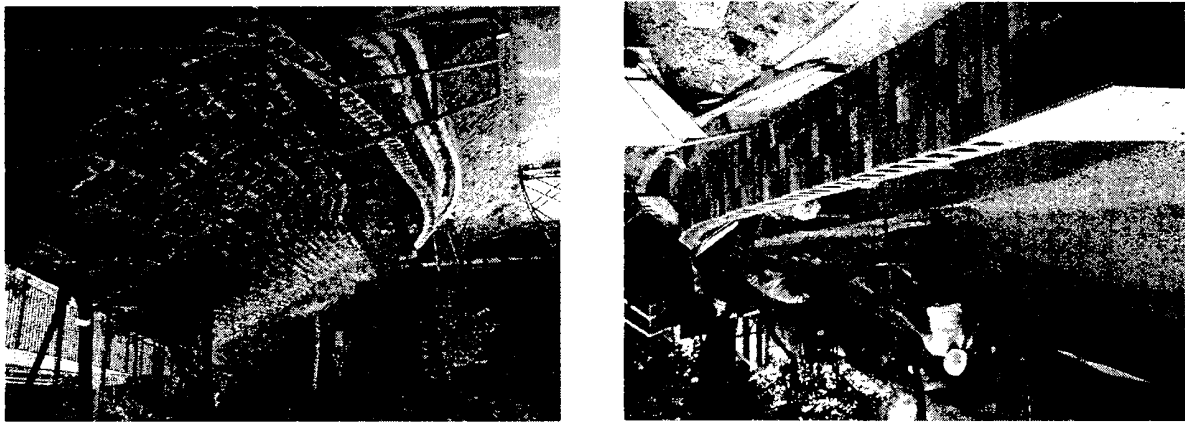
As the reduction of waste generation in construction industry becomes a top priority, the design of slope works has a major role to play. Recently, the design of a major highway project has abandoned the initial choice of using a viaduct to support the link road of the highway in favour of an embankment solution. As illustrated in Figure 8, the viaduct scheme is more vulnerable to natural terrain hazards, which include debris flows and boulder falls. By contrast, the fill embankment can act as a barrier against the potential natural terrain hazards. As the construction of the embankment utilized excavated materials from the adjoining site formation works, the amount of construction waste that needed to be taken off site was significantly reduced.



(a) Viaduct option (more vulnerable to natural terrain hazards)
(b) Embankment option (utilize as barrier against natural terrain)

Figure 8. A comparison of viaduct and embankment options. (Lam & Yim 2001).

Scraped rubber tires, which are processed with cement, fly ash, and elastic binder into either particle form or cemented blocks, are currently available in Hong Kong (Lee et al, 2001). The utilization of this solid waste material for the construction of embankments on soft ground, is advantageous because the light weighted material imposes small surcharge on soft ground, thus allowing a reasonably quick rate of construction without the need of extensive ground treatment works. Using such waste materials for embankment construction not only reduces the need for natural fill materials, it also helps minimizing the disposal of waste materials into landfill sites. With its high porosity, high strength, high stiffness, and good dynamic properties, rubber soil has also been successfully used in a Landslip Preventive Measures (LPM) project for the stabilisation of a loose fill slope (Figure 9).



(a) Placing of rubber soil in progress
(b) Upgraded slope using rubber soil

Figure 9. A fill slope stabilisation works using rubber soil

PROTECTION OF WILDLIFE HABITATS

Hong Kong is home for many different species of birds, such as the rare cliff dwelling birds Peregrine Falcons. A pilot project is currently undertaken at a quarry in the Hong Kong Island to provide habitats for these birds. As illustrated in Figure 10, a man-made falcon nest blasted into a cliff will provide habitats for these birds as well as for the others, creating a balanced ecosystem valuable to the environment. However, before the plan can be finalized, wide consultation among various government departments and concerned groups on the nest design, locations, etc. has to be made.

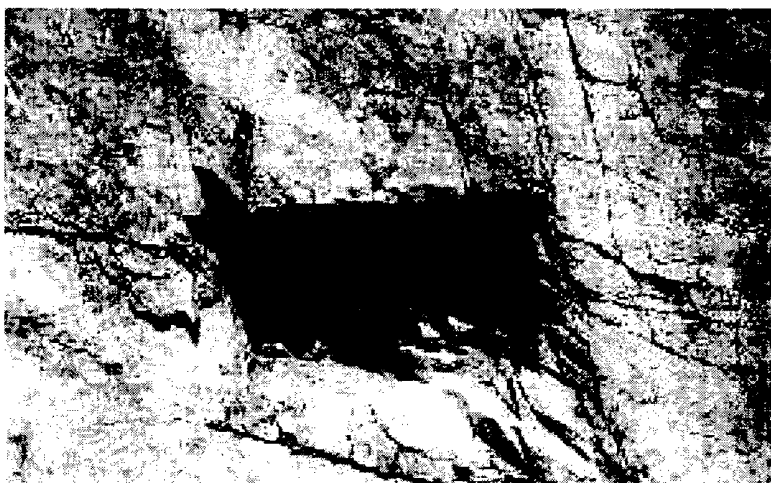
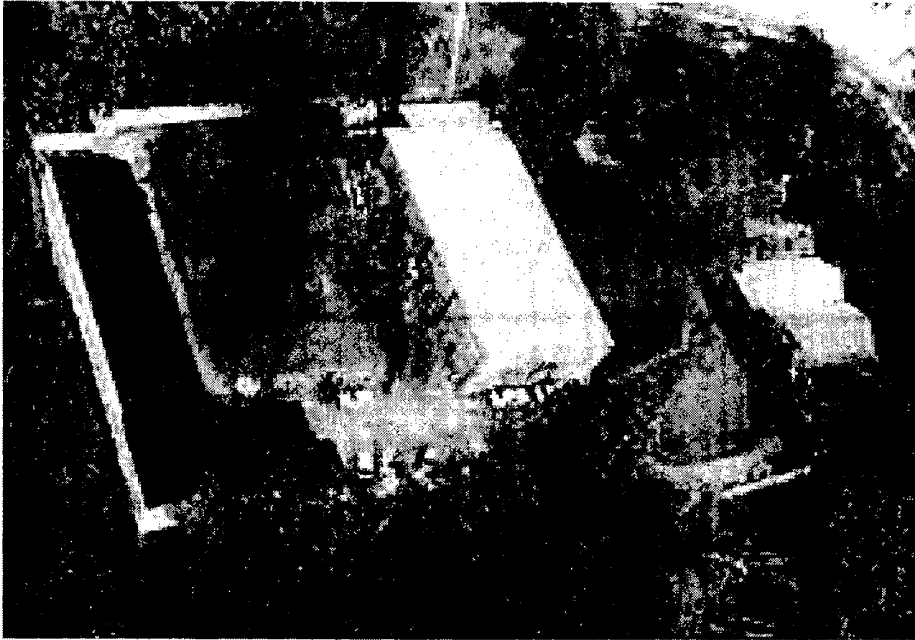


Figure 10. A man-made falcon nest blasted into a cliff

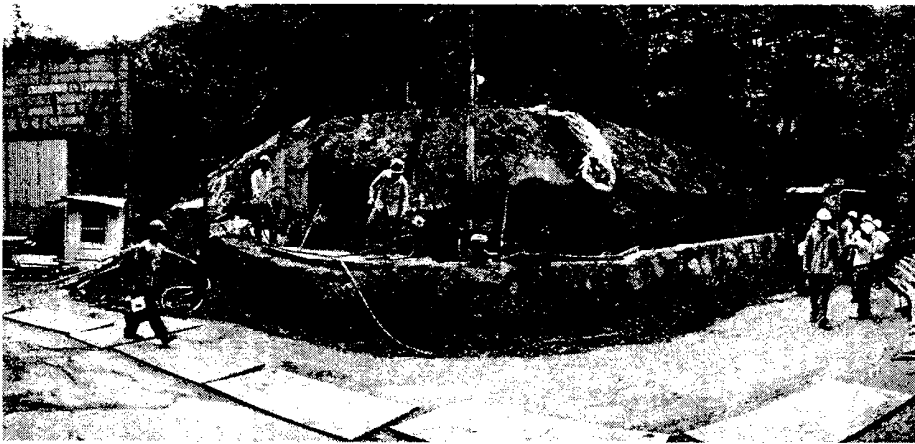
BUILT HERITAGE CONSERVATION

In the short span of a century and a half, Hong Kong has grown from a small rural community into a modern metropolis. The main characteristic of our culture lies to the successful blend of East and West. Many of the Hong Kong's heritage items, which are a testimony to our unique past and are irreplaceable, are buildings, monuments, terraces, masonry walls and slopes. Examples are the old masonry walls and sloping grounds in the Mid-Levels, and even some of the famous landslide sites, such as the Po Shan failure site, and the Tsing Shan Debris Flow Site. These sites are of immense values to the Hong Kong Landslide History. Their conservation provides a legacy for future generations, shows how those before lived, and enables us to retain knowledge and memories of previous ages. Conserved heritage also forms part of our collective memory, which provides a sense of continuity and belonging. Our heritage helps define our cultural identity. Heritage items add variety to our cityscape, and make our city more attractive place in which to live and work. They are also valuable economic assets that are essential elements of Hong Kong's development of cultural tourism.

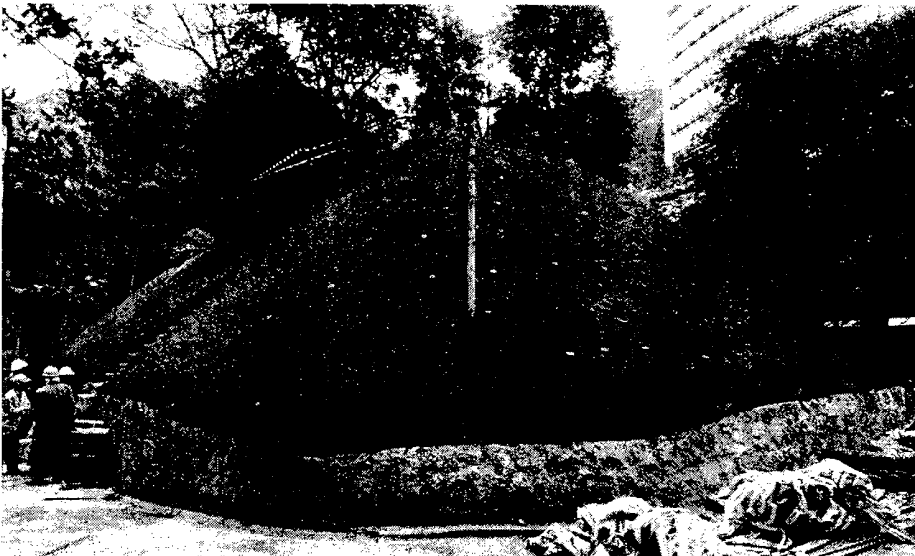
In a redevelopment project of an old British barrack, there are two earth mounds constructed in the pre-war time as a protective measure of the explosive magazine in the barrack against possible air attack or bombing. These mounds were not built to the current safety standards, and removal of the loose filling mounds is required. As these mounds are of high heritage values, this option was objected and not accepted. The project team finally came up with an innovative solution (Figure 11) by installing soil nails in the loose fill bodies as upgrading works, and preserve the structural integrity and the existing form of the earth mounds.



(a) A bird's eye view of the two pre-war earth mounds built as protection measures against air attack or bombing



(b) Installation of soil nails and slope works in progress



(c) Upgraded mound with soil nails concealed under grass cover

Figure 11. Stabilisation works of an earth mound of heritage value.

INCREASING RATE OF RETURN

Slope works is a costly construction activity. Every effort should be made to increase the return of the works, be it tangible or intangible. This is particularly so, as Hong Kong is dealing with less risky slopes today after over two decades of very intensive treatment of Hong Kong slopes. Land is one of the most precious resources in Hong Kong, and making full use of land, be it flat land or sloping ground, is an obvious strategy to take. Some slopes have intrinsic characteristics, such as their size and height, which can provide room/land for various uses. Others are either strategically located or have an abundant supply of groundwater. Most of the slopes have some distinct characteristics that can be turned into “assets”. The authors always promote the generation of innovative ideas to turn slopes into asset, in addition to maintaining stability of slopes (Yim et al, 2002). Examples are slope advertisement (Figure 12), solar energy panels powered irrigation system, grass skiing, Ecological trails, rock carving, slope farming, etc. Many of the ideas can generate revenue, create recreational values, improve environment, promote technology and boost tourism, and help creation of employment opportunities.

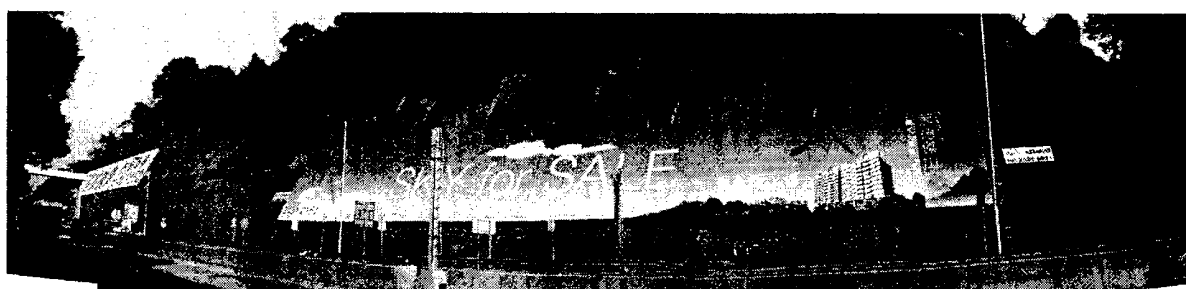


Figure 12. Advertisement on a road side slope.

LIFE CYCLE ASSESSMENT

Life-cycle Assessment/Analysis (LCA) is becoming a popular tool worldwide for assessing the sustainability of buildings. Basically, it helps to evaluate resources (including energy) consumption and environmental impacts incurred by the processes on various stages of the life cycle of the building. Types of LCA impacts considered include Embodied energy (EE), which is related to fossil fuel depletion, CO₂ for global warming, SO₂ for acid rain, and Dust for air pollution. It facilitates building designers in making decisions on alternative options in material use as well as building services systems.

In Hong Kong, adoption of LCA in building design is still at a developing stage. We are facing the difficulty of having reliable LCA model and local database for performing credible assessment. Recent private and public initiatives on the LCA development will help bring this technique into the mainstream practice (Cheng, 2004).

The life cycle of a slope feature is relatively simpler than that of a building. It would also be easier to establish a LCA model for slope feature. Using “Simple Payback” analysis and simple indicators, such as, embodied energy and Oversea established Eco-indicators, it is possible to conduct a quick and simple LCA for slope works. Figure 5 shows an example how embodied energy could be used for evaluating the sustainability impacts of reinforced fill wall and reinforced concrete wall. The geotechnical profession should work with other disciplines to gain knowledge and experience in the use of the LCA. With time, LCA will soon become a powerful tool for evaluating the sustainability impacts on slope works.

CONCLUSIONS

Based on the concept of sustainable development, a set of sustainability considerations was identified and had been adopted in the design of slope works in civil engineering and building projects in Hong Kong.

Experience shows that visual harmony can be achieved through short-term and long-term aesthetics treatment, and ecological enhancement will often benefit the society, economy and the environment. Slope works designed on the sustainability basis are having less impact on the environment, with higher rate of return and are more cost effective.

Life Cycle Assessment (LCA) is becoming a powerful tool for evaluating sustainability impacts on buildings as well as slope works.

REFERENCES

- Fong, W F K & Yeung, J S K (2003). *Production and Application of Recycled Aggregates*. Proceedings of the Seminar on Green Buildings, Hong Kong Institution of Engineers, 24 Feb 2003, Hong Kong pp 39-47.
- Lam, A Y T & Yim, K P (2001). *Sustainable Geotechnical Design Works for the Tsing Yi North Coastal Road Project, Hong Kong*. Proceedings of the 14th Southeast Asian Geotechnical Conference, 10-14 Dec 2001, Hong Kong. Vol. 1, pp 373-378.
- Lam, A Y T, Lau, K.W. K. & Lee, C.K.T. (2001). *Design and Construction of a 16m High Reinforced Fill Retaining Wall in Hong Kong*. Proceedings of the 14th Southeast Asian Geotechnical Conference, 10-14 Dec 2001, Hong Kong. Vol. 1, pp 361-366.
- Lee, K M, Cheung, B K W, Yip, C L Y & Zhu, G (2001). *The Use of Recycled Scrap Rubber Tires for Construction of Earth Structures*. Proceedings of the Seminar on Environmentally Friendly Structures, Hong Kong Institution of Engineers, 22 May 2001, Hong Kong, pp 66-80.
- Mak, N S C (2003a). *Geofiber Slopes in Hong Kong*. Hong Kong Institution of Engineers Transaction. Vol. 10, No. 1, March 2003, pp 21-25.
- Mak, N S C (2003b). *Technical Presentation at the Seminar on Green Slope technique*, organized by the Environmental Division of the Hong Kong Institution of Engineers, on 26 August 2003.
- World Commission on Environment and Development (1987), *"Our Common Future"*, Oxford University Press, 1987.
- Cheng, V. (2004). Technical presentation, *"Development of Life Cycle Assessment (LCA) Tools for Commercial Buildings in Hong Kong"* at the Technical Meeting organised by the Environmental Division of the Hong Kong Institution of Engineers, on 19 Feb 2004.
- Yim, K P & Chan, D Y W (2003). *A Case History of Simple Solution to a Complex Geotechnical Engineering Problem*. Proceedings of the Hong Kong Institution of Engineers Annual Seminar on Case Histories in Geotechnical Engineering in Hong Kong. 9 May 2003, Hong Kong, pp 305-318
- Yim, K P, & Lam, A Y T (2002). *Hong Kong's Slopes Towards Sustainability*. Proceedings of the International Conference on Innovation and Sustainable Development of Civil Engineering in the 21st Century. 1-3 August 2002, Beijing, pp 37-40.
- Yim, K P, Lau, A S T, Kwok, D W & Lam, A Y T (2002). *Thoughts on Turning Slopes into Asset*. Proceedings of the International Conference on Innovation and Sustainable Development of Civil Engineering in the 21st Century. August 2002, Beijing, pp 221-224.

ACKNOWLEDGEMENTS

This paper is published with the permission of the Director of Architectural Services of the Hong Kong Special Administrative Region.

SOIL NAIL HEAD REVIEW

Y. K. Shiu¹ & Garry W. K. Chang¹

Abstract: A study on the role and behaviour of soil nail heads has been carried out. The study includes a review of the fundamental mechanism of soil nail heads and the international practice in nail head design. Numerical simulations have also been undertaken to examine analytically the effects of nail heads on the stability of nailed slopes and bearing failure of nail heads in soil. Results of the study show that soil nail heads can enhance the contribution of soil nails to the stability of slopes, and also that bearing capacity of soil at nail heads needs to be considered in design. The paper reports and discusses the findings of the study.

INTRODUCTION

Soil nails are used extensively for slope improvement works in Hong Kong. To permit planting of vegetation on slopes, soil nail heads of recent slope designs are usually in the form of isolated concrete pads. A study has been carried out to examine the role of this type of nail heads.

The study comprises a review of the fundamental mechanism of soil nail heads and technical guidance documents from UK, USA, France, Japan and Germany in respect of design practice of soil nails. In addition, a number of numerical simulations using the finite difference code, FLAC (Fast Lagrangian Analysis of Continua), have been carried out to investigate the functions of soil nail heads and examine the bearing failure of nail heads in soil.

LOAD TRANSFER MECHANISM OF SOIL NAIL HEADS

The soil in a nailed structure consists of two distinct zones, namely the active zone and the resistant zone as shown in Figure 1.

The major load transfer mechanism of a nailed structure is the development of tensile forces in the nails as a result of the restraint that the nails and the nail heads provide in response to deformation of the structure. In the case of stabilization of a marginally stable slope, the deformation is associated with movement of the slope as a result of inadequate support. In the case of a nail-supported excavation, the deformation is due to movement of the retained ground as well as expansion of the reinforced zone as a result of removal of lateral constraint as excavation proceeds. In either case, movement of the soil relative to the nails generates longitudinal shear stress along the soil/nail interface, which in turn induces tension in the nails and generates reaction from the nail heads. The soil movement also induces bending moment and transverse shear stress in the nails. Support to the ground mainly comes from the tension in the nails whilst the shear force and the bending moment contribute little to the support.

Soil nails are usually placed at inclinations close to the horizontal. These inclinations correspond in general with those of maximum tensile strains developed in the soil in the active zone. As such, the predominant action of the nails is in tension (Jewell & Wroth

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(1987), Hayashi et al (1988)). The interaction between the nail head and the soil gives rise to tensile load at the head of the nail.

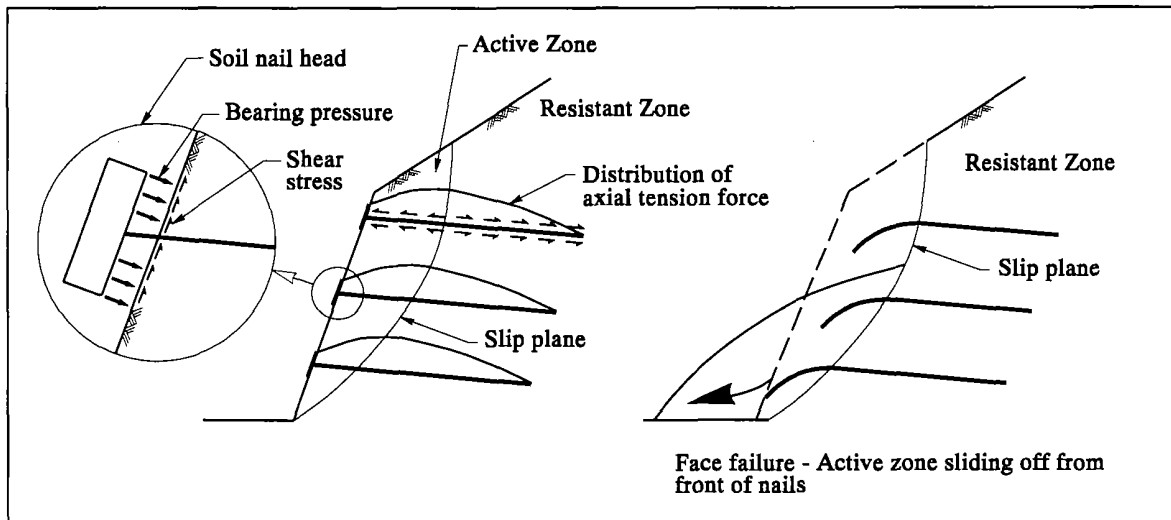


Figure 1 - Load Transfer Mechanism of Soil Nailed Structure

The soil nails and the nail heads act together to tie the active zone to the resistant zone. The nail head and the pullout resistance of the nail in the active zone must be adequate to provide the required nail tension at the slip surface between the active and resistant zones in order to prevent the active zone from sliding off the soil nail. This is illustrated in Figure 1. For stability to be achieved, the soil behind isolated nail heads must have sufficient strength against bearing failure in the soil.

A number of studies on soil nail heads including model tests (e.g. Muramatsu et al (1992)), full-scale field studies (e.g. Gässler & Gudehus (1981), Stocker & Riedinger (1990)), centrifuge model tests (e.g. Tei et al (1998)) and numerical simulations (Babu et al (2002)) have been performed at different parts of the world. Results of these studies show that soil nail heads, whether in the form of individual concrete pads or as part of concrete facing, play a significant role in enhancing the shearing resistance of the soil in the active zone.

SOIL NAIL HEAD DESIGN METHODS

The design methods as used in Hong Kong, UK, USA, France, Japan and Germany are summarized below, drawing attention to common similarities and relevant differences. Some of the design methods are related mainly to nailed retaining walls where the nail heads are integrated into a continuous reinforced concrete facing. As such, they mostly deal with the determination of the structural strength of the nail heads (e.g. flexural strength and punching shear strength) and the nail-facing connections. In some countries (e.g. UK and Japan), guidance is also given for the design of isolated nail heads whose capacities are also controlled by bearing failure in soil.

Hong Kong Approach

There is no standard for the design of soil nail heads in Hong Kong. Watkins & Powell (1992) proposed that the soil beneath the nail head should have a minimum factor of safety of 3 against bearing failure resulting from the load induced by the nail head. No details were given as how the bearing capacity should be determined.

Standard nail head size of 400 mm x 400 mm x 250 mm thick is commonly provided in

the Landslip Preventive Measures (LPM) design. The standard nail head size is used prescriptively and bearing capacity check is in general not carried out in design.

For private projects, most practicing engineers base their designs on the presumed allowable bearing capacity of soils for foundation on horizontal ground given in the PNAP 141: Foundation Design by the Buildings Department of the HKSAR. Some engineers use the approach stipulated in HA68/94 of the U.K. (Department of Transport, 1994); and some others use bearing capacity equations, assuming that the nail heads are founded on horizontal ground.

United Kingdom Approach

Methods for design of nailed structures and reinforced fill structures can be found in HA68/94. The design is based on limit state principles incorporating partial factors. According to HA68/94, if the soil nails are not connected to a continuous facing but to a 'waling plate', the bearing capacity of the front face 'waling plate' should be checked.

The bearing force acting on the slope surface by the nail head is taken as the difference between the designed tension in the nail and the soil/nail frictional force developed along the nail length at the active zone. A lower-bound solution for checking the bearing capacity of the 'waling plate' in soil is provided and it is shown in Figure 2. The bearing capacity of a nail head is estimated based on the angle of shearing resistance (ϕ') of the soil whereas the cohesion is not considered. The dimensions of the nail head can be determined using this solution.

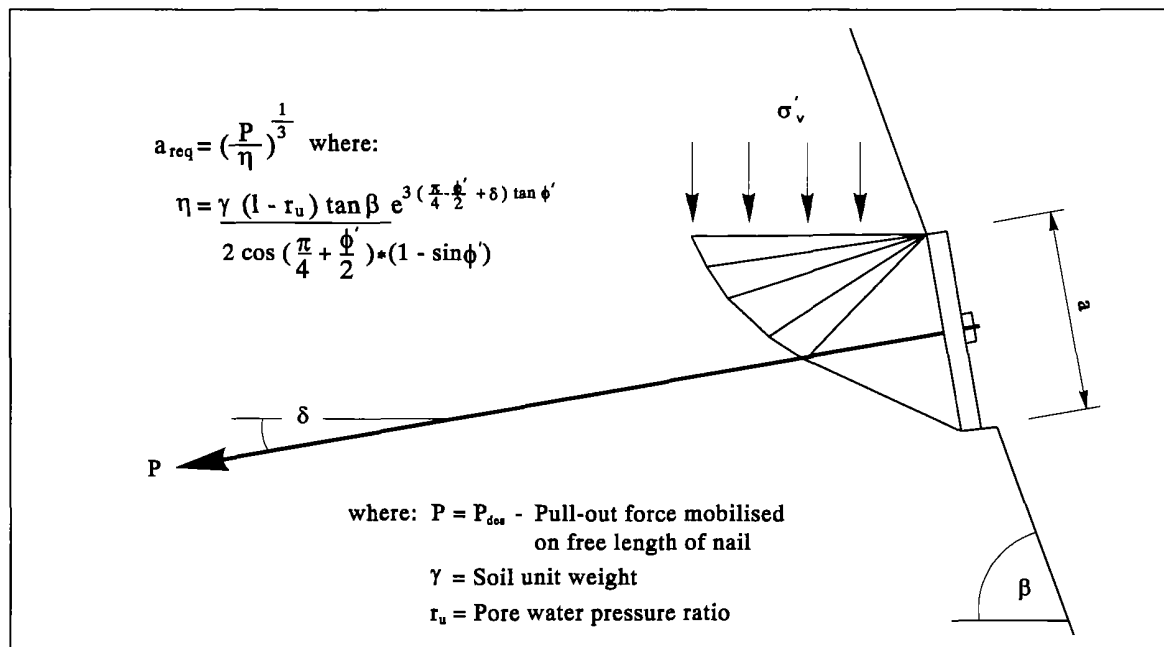


Figure 2 - Nail Plate Bearing Capacity (after HA 68/94)

United States Approach

The recommended soil nail design method is given in FHWA (1998). The design recommendations are primarily related to nailed retaining walls and as such mainly nail heads attached to a facing are considered. This method considers directly the influence of the facing and the magnitude and distribution of nail forces. In this approach, a trial pattern of soil nails is first selected. An allowable nail load support diagram is then developed based on factored nail head resistance, factored pull-out resistance and the allowable tensile strength

of the nail (see Figure 3). The nail forces contributing to the overall stability of the nailed soil mass are determined at the location where the failure plane intersects the nail load support diagram.

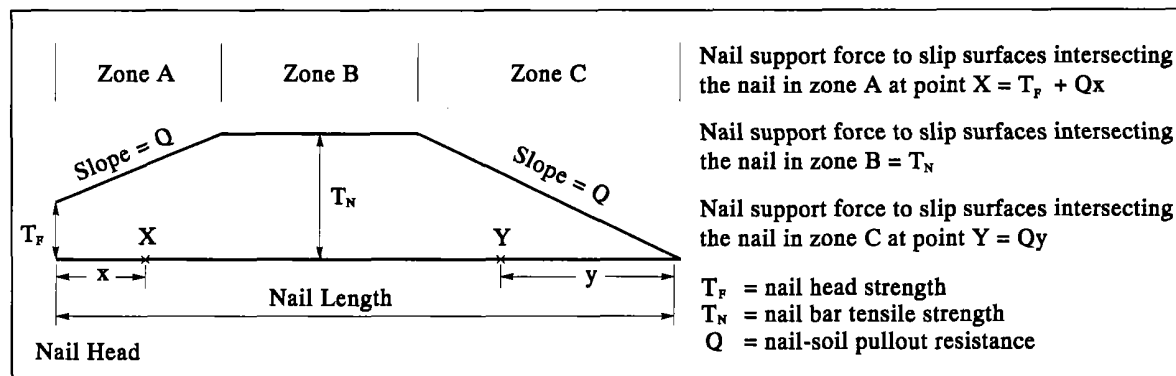


Figure 3 - Allowable Nail Load Support Diagram (after Joshi, 2003)

It can be noted from Figure 3 that the strength of nail head (T_F) is an important parameter for developing the nail load diagram that gives the magnitude and distribution of tensile forces along the soil nail. It is one of the elements required to define the overall reinforcing capacity of the nail. FHWA (1998) emphasizes that the distribution of pressure on the facing between the nails is non-uniform and arching effects tend to develop horizontally and vertically between the nails, and as a result, stresses are concentrated at the nail heads. The higher pressure at the nail head needs to be considered in design. A nail head load corresponding to 50% of the maximum nail load should be adopted for design purposes.

French Approach

An approach of soil nail design is given in French National Research Project (1991). As the design approach is mainly for nailed retaining walls, method for design of facing rather than isolated nail head is provided in the document. The facing is designed on the basis of a uniform earth pressure corresponding to a fraction of the maximum tension T_{max} that can be developed in the soil nail. It requires the determination of the tension, T_o , at the soil nail heads. The values of the ratio T_o / T_{max} depend on the horizontal spacing (S_h) and the vertical spacing (S_v) of the soil nails. The maximum value of this ratio can be obtained from the following empirical equations:

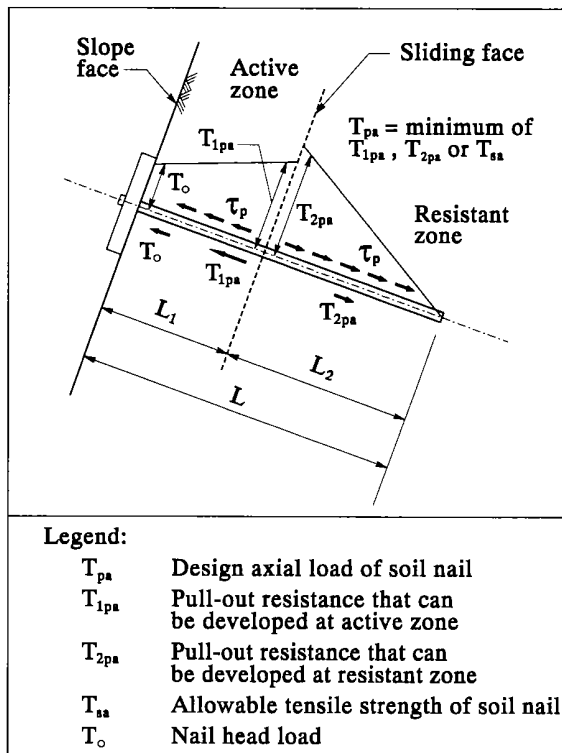
$$\begin{aligned} T_o / T_{max} &= 0.5 + (S - 0.5) / 5 && \text{when } 1 \leq S \leq 3 \text{ m} \quad \dots (1) \\ T_o / T_{max} &= 0.6 && \text{when } S \leq 1 \text{ m} \\ T_o / T_{max} &= 1 && \text{when } S \geq 3 \text{ m} \end{aligned}$$

where S = maximum of S_v and S_h , expressed in meter.

Same as the US approach, the French National Research Project (1991) points out that the distribution of pressure on the facing between soil nails is non-uniform because arching effects tend to develop between the nails and this results in stress concentrations in the vicinity of the nails.

Japanese Approach

The design approach used in Japan directly considers the earth pressure acting on the nail head (T_o) when calculating the distribution of tension force in the soil nails. According to



Miki et al (1997), the design axial load of a soil nail, T_{pa} , should be the lowest among: (i) pull-out resistance that can be developed at the active zone, T_{1pa} ; (ii) pull-out resistance that can be developed at the resistant zone, T_{2pa} ; and (iii) allowable tensile strength of the soil nail, T_{sa} . Figure 4 depicts the concept. It also shows that the pull-out resistance in the active zone will be very small if the force at soil nail head (T_o) is not considered.

For the soil nail head design load, Japan Highway Public Corporation (1998) suggests that a force coefficient, $\mu = T_o / T_{max}$, is to be used to determine the nail head load T_o . The coefficient is based on measurements of actual nailed structures and model tests. It lies between 0.2 and 1.0, depending on the nail spacing, nail length and the type of surface where the nail heads rest on.

Figure 4 - Nail Force Distribution (after Miki et al, 1977)

German Approach

According to Stocker & Riedinger (1990), the standard German practice is that shotcrete linings are designed for 85 % of the active Coulomb's earth pressure at the back face of the wall.

Discussion

The overseas design methods described above illustrate that the capacity of nail heads to develop tension in soil nails is an important design consideration.

In all the overseas methods, the size, thickness and reinforcement details of soil nail heads are determined on the basis of the earth pressure acting on the nail heads. Two main design aspects are considered; they are the bearing capacity of the soil beneath the nail head and the structural strength of the nail head itself. Many of the design methods (such as those used in France and U.S.) are developed mainly for nailed walls where the nail heads form part of the concrete facing. In those cases, bearing failure of the nail head or facing is unlikely to occur and as such little guidance has been provided in respect of bearing capacity evaluation. A method on design against bearing failure of the soil behind isolated nail heads is given in the UK guidance document HA68/94.

NUMERICAL ANALYSES

Numerical simulations were carried out using the two-dimensional finite difference code, Fast Lagrangian Analysis of Continua (FLAC), which was developed by Itasca (1996). A

Mohr Coulomb model was used for the soil. The soil nail was modeled as cable elements. Soil-nail interaction was represented by a spring-slider system having shear springs located at the nodal points along the cable elements.

The numerical simulations are divided into two principal parts: (a) investigation of the effect of nail heads on stability of nailed slopes; and (b) determination of the sizes of pad nail heads based on bearing capacity consideration.

Effect of Nail Heads on Stability of Nailed Slopes

Two cases were considered in the analysis: (i) unreinforced slope, and (ii) slope reinforced with soil nails. For both cases, the model slopes are 20 m high, standing at an angle of 55°, and with an up-slope of 10° in gradient at the crest. For the case of reinforced slope, seven rows of soil nails were provided. This corresponds to a vertical nail spacing (S_v) of 2 m. The horizontal spacing of the nails (S_h) is taken to be 1.5 m. Each nail is 20 m long with a 40 mm diameter steel bar in a 100 mm grouted hole. The shear strength parameters of the soil were assumed to be $c' = 10$ kPa, and $\phi' = 43^\circ$, so that the unreinforced slope had a factor of safety of 1. Figure 5 shows the geometry of the slope and the material parameters used in the numerical analysis for the reinforced slope. The model for the unreinforced slope is the same as that shown in the figure except without the soil nails and nail heads.

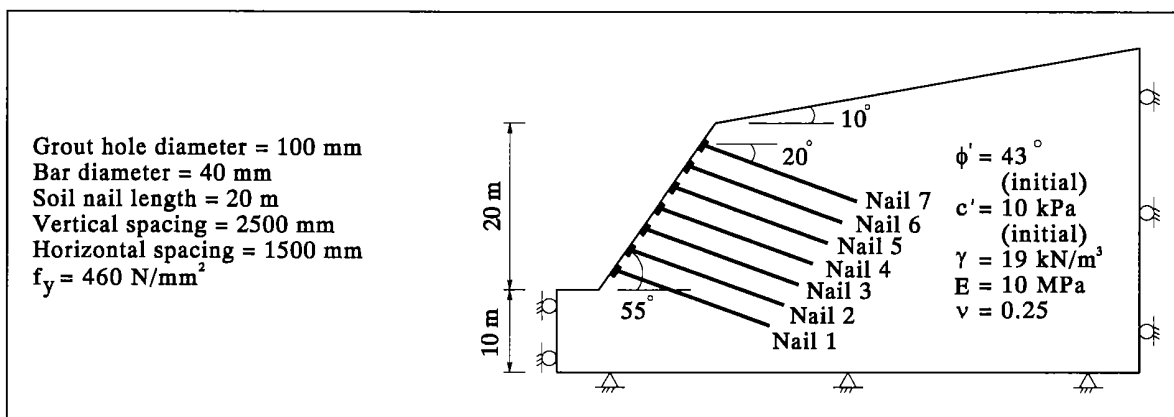


Figure 5 - Geometry and Material Parameters of Model Slope

A Mohr Coulomb model was used for the soil. The nail heads and facing were modeled using the linear elastic strength criterion. A cable element was used to represent the soil nail as the bending stiffness of the soil nail was not considered. Developments of the tensile forces in the nails were governed either by the tensile strength of the nail or the peak shear strength at the soil-grout interface.

Since the FLAC adopts 2-dimensional plain strain analysis, the nail heads were modeled as infinitely long beams of specific widths. In order to investigate the influence of the size of the nail heads on the stability of slopes, two nail head sizes were considered. They were 400 mm wide and 800 mm wide. All of them were 250 mm thick. In addition, continuous concrete facing was also studied.

Analytical Approach

Slope stability analysis was first carried out on the unreinforced slope. From the results of the analysis, the unreinforced slope had a minimum factor of safety (FoS) equal to 1.0 for the initial soil strength parameters of $c_i' = 10$ kPa, and $\phi_i' = 43^\circ$.

In slope engineering, the FoS is conventionally defined as the ratio of the actual soil shear strength to the minimum shear strength required for equilibrium. As pointed out by Duncan (1996), FoS can also be defined as “the factor by which the shear strength of the soil would have to be divided to bring the slope into a state of barely equilibrium”. FoS can therefore be determined simply by reducing the soil shear strength until failure occurs. This strength reduction approach is often used to compute FoS using finite element or finite difference programs (Dawson et al (1999), Krahn (2003)). In this study, the approach has also been adopted to determine the FoS of slopes for the following cases:

- (a) nailed slope with no nail heads,
- (b) nailed slope with 400 mm wide strip nail heads,
- (c) nailed slope with 800 mm wide strip nail heads, and
- (d) nailed slope with nail heads in the form of a full surface facing of 250 mm thick.

For each case, the FoS was determined by progressively reducing the shear strength of the soil in the model until numerical non-convergence of unbalanced forces and displacements at chosen monitoring points occurred. This analysis was done by trial and error using parameters c_m' and ϕ_m' , where

$$c_m' = c_i' / \text{FoS} = 10 / \text{FoS} \quad (\text{kPa}) \quad \dots\dots\dots (2)$$

$$\phi_m' = \tan^{-1} (\tan \phi_i' / \text{FoS}) = \tan^{-1} (\tan 43^\circ / \text{FoS}) \quad \dots\dots\dots (3)$$

Results of Numerical Simulations

The relationship between the calculated FoS and the nail head sizes for the model slopes, as obtained from the FLAC analyses, is plotted in Figure 6. The FoS increases from 1.0 for the unreinforced slope to 1.2 for the nailed slope with no nail heads. Substantial increases in the FoS are obtained with nail head sizes increasing from 400 mm wide to 800 mm wide. The trend of increase levels off for nail head sizes larger than 800 mm wide. It shows that nail head has significant effect on the stability of a soil nailed slope.

The total of the maximum tensile forces mobilised in all the soil nails ($\Sigma T_{\max}$) at limit equilibrium condition of the model are plotted in Figure 7. The figure shows that $\Sigma T_{\max}$ increases with the increasing size of the nail heads. Close similarity between Figure 6 and Figure 7 indicates that the FoS of the nailed slopes are directly related to $\Sigma T_{\max}$.

Figure 8 compares the distribution and magnitude of the axial forces developed in the soil nails for the reinforced slope with no nail head and the reinforced slope with nail head of 800 mm wide. For the slope with no nail head, no tension forces are induced at the front end of the nails and the maximum tension ($T_{\max}$) occurs close to the slope face. For the slope with 800 mm wide nail head, the force distribution pattern is markedly different from that with no nail head. Substantial tension force was generated at the nail head (T_o), and the maximum tension ($T_{\max}$) occurred at a distance behind the nail head.

Figure 9 shows the distribution of the ratio of $T_o / T_{\max}$ for nailed slopes with different nail head sizes. The values for $T_o / T_{\max}$ range mainly between 0.55 and 0.75.

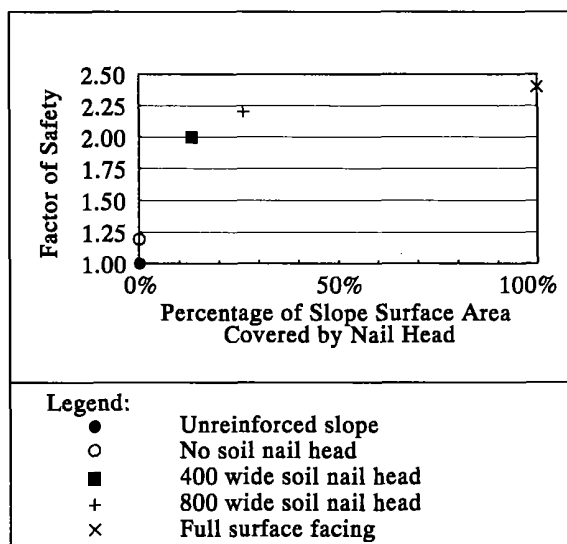


Figure 6 - Relationship between Factor of Safety and Nail Head Size

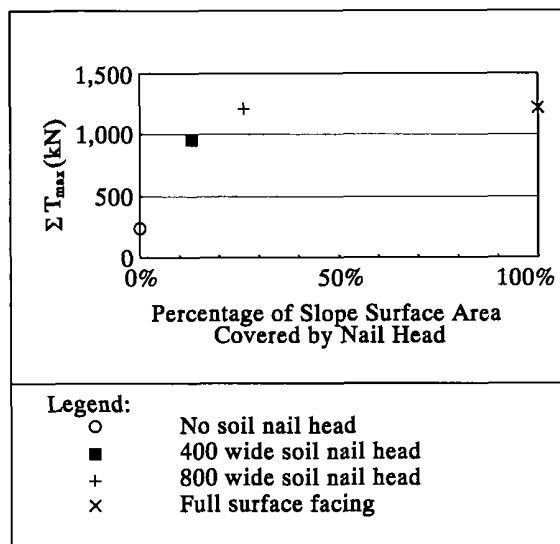


Figure 7 - Relationship between Total Maximum Tensile Forces and Nail Head Size

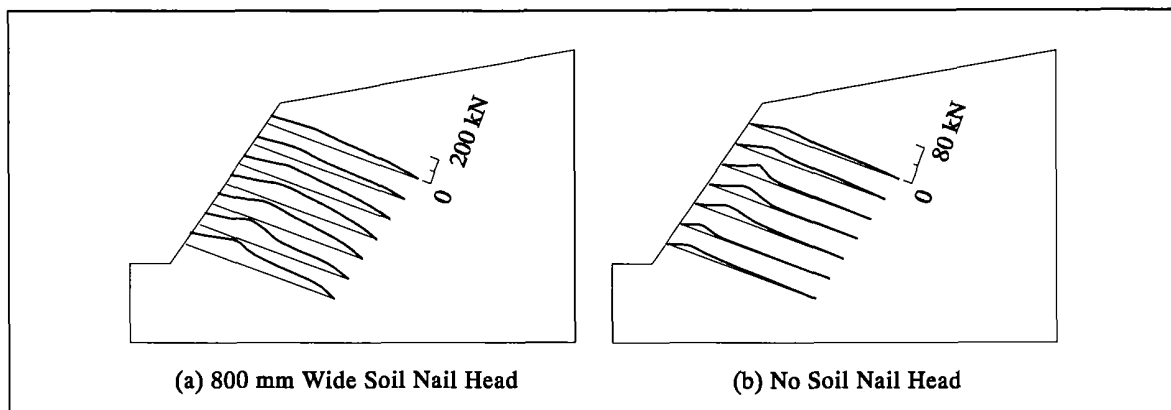


Figure 8 - Variation of Axial Nail Forces for (a) 800 mm Soil Nail Head and (b) No Nail Head

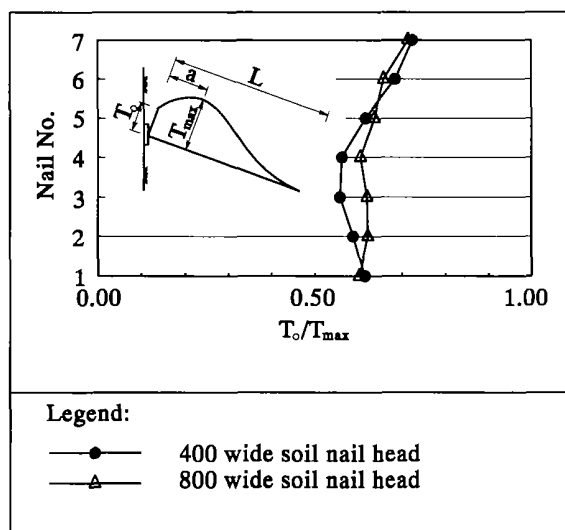


Figure 9 - Distribution of T_o/T_{max} in Soil Nails

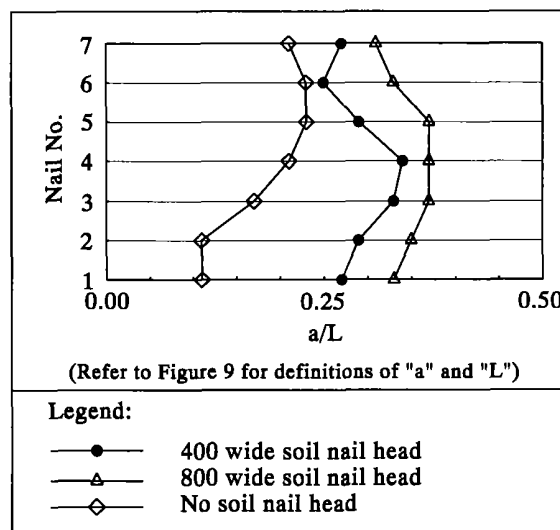


Figure 10 - Locations of Maximum Tensile Force in Soil Nails

Figure 10 shows the locations of the maximum nail forces developed in the soil nails for the reinforced slopes with different nail head sizes at the limit equilibrium condition. The locations of T_{max} in general lie further away from the slope face for wider nail heads. As the locus of the T_{max} in a nailed slope at limit equilibrium corresponds with the failure plane, larger nail heads tend to force the failure planes further away from the slope face. This agrees with the observations made by Gutierrez & Taksuoka (1988).

Results of the numerical simulations illustrate that soil nail heads can have significant beneficial effects on the stability of nailed slopes.

Determination of Nail Head Sizes Based on Bearing Capacity Consideration

FLAC analysis has been performed to examine bearing capacity failure of square soil nail heads. A small slope model of 5 m in height was used in the analysis. Three slope angles were considered: 45°, 55° and 65°. Typical details of the model used are shown in Figure 11. In the analysis, the nail head was pushed into the soil face by a nail force (T_o) to simulate the situation of soil moving out from a slope and pressing against a soil nail head. The nail head was placed at the lower part of the small model slope. This was based on the result of a sensitivity analysis on the position of the nail head, which showed that bearing failures were sensitive to the vertical extent of the slope above the nail head but insensitive to the extent of the slope below the nail head.

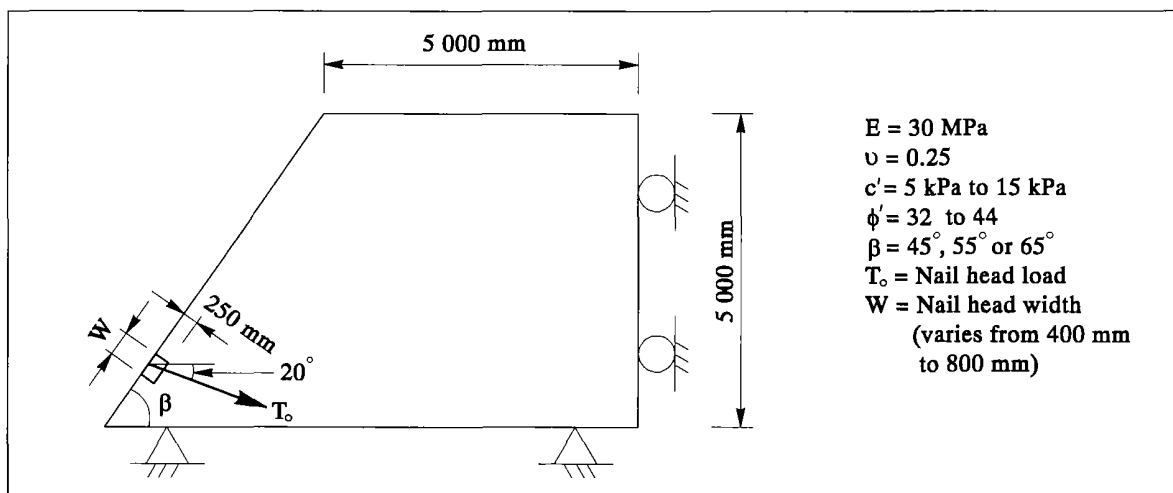


Figure 11 - Details of Slope Model for Bearing Failure Analysis

The nail forces used were determined from the allowable tensile strength of steel bars (f_m), using the following equations:

$$T_o = f_m A_s \text{ (kN)} \quad \dots\dots\dots (4)$$

$$f_m = 0.55 f_y \leq 0.23 \text{ (kN / mm}^2\text{)} \quad \dots\dots\dots (5)$$

where f_y is the characteristic strength of a high yield bar, and A_s is the cross-sectional area of the steel bar.

The values of T_o vary directly with the cross-sectional area of the steel bar. The nail load T_o calculated in this way is based on a simplifying assumption that $T_o = T_{max}$.

The nail force was applied at the center of the nail head at an inclination of 20° from the horizontal. Bar sizes commonly used in soil nail design were considered. As the current corrosion measures require the provision of a 2 mm sacrificial thickness on the bar radius, the reduced bar sizes resulting from this corrosion were also considered. This means a smaller

value of T_0 .

Simulations were performed for three nail head sizes: 400 mm x 400 mm, 600 mm x 600 mm and 800 mm x 800 mm. The nail heads were all 250mm thick and modelled as an elastic material. Since it was a plain strain analysis, T_0 was adjusted as the force per meter run based on the side length of the nail head. For a given nail head size and a given T_0 , the strength parameters of the model slopes were varied until numerical non-convergence of unbalanced forces and displacements occurred. A common range of shear strength parameters for saprolithic soils was considered in the simulations.

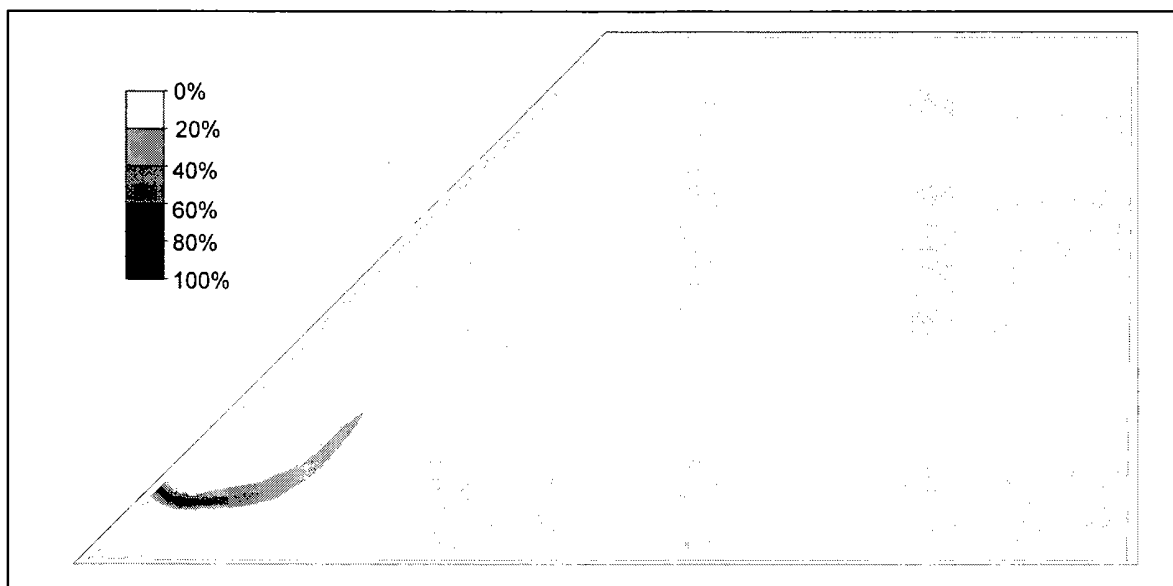


Figure 12 - Typical Shear Strain Vectors Plot

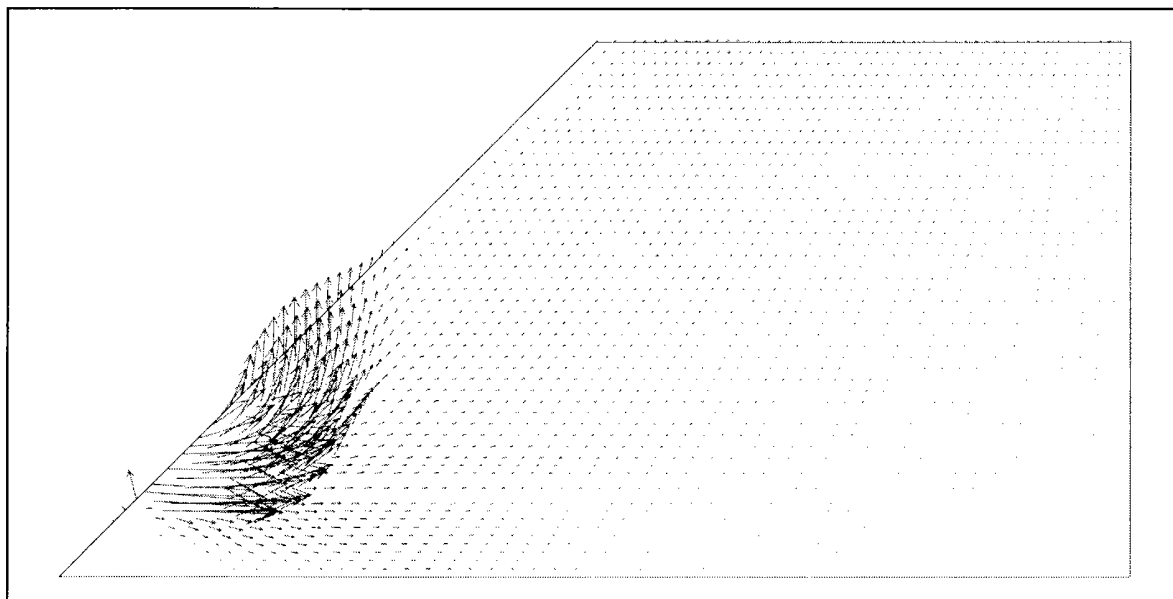


Figure 13 - Typical Displacement Vectors Plot

Results of Analyses

Figures 12 and 13 show respectively the shear strain vectors and displacement vectors at the point of bearing failure for a 600 mm x 600 mm nail head on a 45° slope. Nail heads of other sizes also have the same failure mode which is similar to that considered in HA68/94 (see Figure 2).

Typical results of the analyses in terms of c' - ϕ' envelope for limiting equilibrium (i.e. bearing failure occurs) for the 600 mm x 600 mm nail head installed on a 45° slope are plotted in Figure 14. In the plot, the nail head loads are expressed as diameters of steel bars. The limit stability line shown in the Figure sets a boundary below which the slope models become numerically unstable even without the application of nail head force.

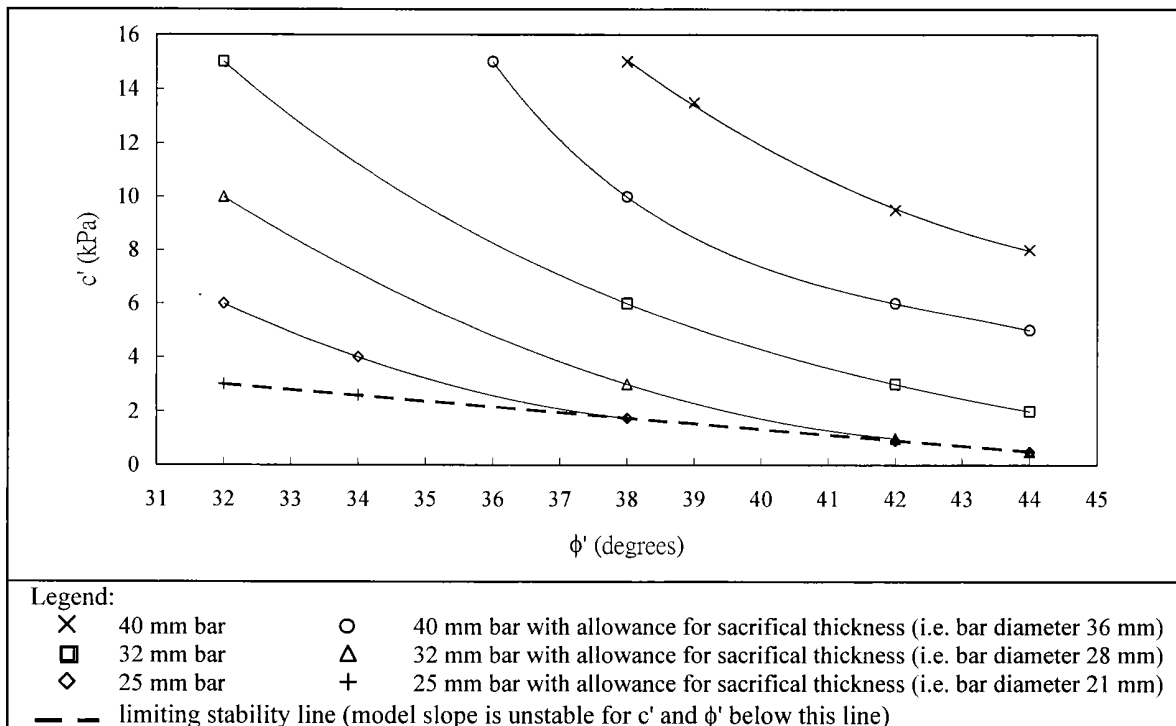


Figure 14 - Soil Strength Envelopes for 600 mm x 600 mm Nail Head at 45° Slope Angle

It should be noted that the bearing capacity of a soil head is related to the angle of the slope, with lower bearing capacity for smaller slope angles. This is because the overburden pressure at the front section of the soil nails is lower for less steep slopes.

CONCLUSIONS

The effect of soil nail head on stability of slopes has been examined analytically in this study. The results of the analysis together with overseas research results and experiences attest to the benefits of soil nail heads to slope stability. Soil nail head works as a reaction pad for the mobilization of tensile force in the soil nail. For large soil nail heads, the tensile force at the nail heads can also confine the soil immediately behind the head. This effect can significantly increase the stability of the soil in the active zone.

Results of the numerical simulations show that bearing failure of nail heads in soil is a failure mode that needs to be considered in the design of soil nails.

REFERENCES

- Babu, G.L.S., Murthy, B.R.S. & Srinivas, A. (2002). "Analysis of Construction Factors Influencing the Behaviour of Soil-Nailed Earth Retaining Walls." *Ground Improvement*, 6, No. 3, 137-143.
- Dawson, E.M., Roth, W.H. & Drescher, A. (1999). "Slope stability analysis by strength reduction". *Géotechnique*, Vol. 49, No. 6, 835-840.
- Department of Transport (1994). *Design Manual for Roads and Bridges: Design Methods for the Reinforcement of Highway Slopes by Reinforced Soil and Soil Nailing Techniques*. HA68/94, Department of Transport, UK.
- Duncan, J.M. (1996). "State of the art: limit equilibrium and finite element analysis of slopes." *Journal of Geotechnical Engineering*, ASCE 122, No. 7, 577-596.
- FHWA (1998). *Manual for Design & Construction monitoring of Soil Nail Walls*. Federal Highway Publication No. SA-96-069R, U.S. Department of Transportation, Federal Highway Administration, Washington, D.C.
- French National Research Project (1991). *Recommendations Clouterre: Soil Nailing Recommendations for Designing, Calculating, Constructing and Inspecting Earth Support Systems Using Soil Nailing (English Translation)*. Presses de lecole Nationale des Ponts et Chaussees.
- Gässler, G., & Gudehus, G., 1981. "Soil Nailing - Some Aspects of a New Technique." *Proceedings of Tenth International Conference on Soil Mechanics and Foundation Engineering*, Vol. 3, Session 12, Stockholm, 665-670.
- Gutierrez V. & Tatsuoka, F. (1988). "Roles of Facing in Reinforcing Cohesionless Soil Slopes by Means of Metal Strips." *Proceedings of the International Geotechnical Symposium on Theory and Practice of Earth Reinforcement*, IS Kyushu '88, Fukuoka, Balkema, Rotterdam, 289-294.
- Hayashi, S., Ochiai, H., Yoshimoto, A., Sato, K. & Kitamura, T. (1988). "Functions and effects of reinforcing materials in earth reinforcement." *Proceedings of International Geotechnical Symposium on Theory and Practice of Earth Reinforcement*, IS Kyushu '88, Fukuoka, Balkema, Rotterdam, 99-104.
- Itasca (1996). *Fast Lagrangian Analysis of Continua (FLAC) Manual, Version 4.0*. Itasca Consulting Group, Inc., Minnesota.
- Japan Highway Public Corporation (JH) (1998). *Design & Works Outlines on the Soil-Cutting Reinforcement Soilworks*. Japan Highway Public Corporation.
- Jewell, R.A. & wroth, C. P. (1987). "Direct Shear Tests on Reinforced Soil." *Geotechnique*, Vol. 37, No. 1, 53-68.
- Joshi, B. (2003). "Behaviour of Calculated Nail Head Strength in Soil Nailed Structures." *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, September, 819-828.
- Krahn, J. (2003). "The 2001 R.M. Hardy Lecture: The limits of limit equilibrium analyses." *Canadian Geotechnical Journal*, Vol. 40, 643-660.
- Miki, H., Fukuda, N., Murata, O., Tayama, S., Yamamoto, A., Otani, Y. & Yokota, Y. (1997). "Present State and Perspective of Design Methods of Reinforced Soil Structures." *Earth Reinforcement*, (edited by H. Ochiai, N. Yasufuku & K. Omine), Balkema, Rotterdam, 1147-1158.
- Muramatsu, M., Nagura, K., Sueoka, T., Suami, K. & Kitamura, T. (1992). "Stability analysis for reinforced cut slopes with facing." *Proceedings of the International Symposium on Earth Reinforcement Practice*, (edited by H. Ochiai, S. Hayashi & J. Otani) Fukuoka, Kyushu, Japan, 11-12 November, 503-508.
- Stocker, M.F. & Riedinger, G. (1990). "The Bearing Behaviour of Nailed Retaining Wall." *Proceedings of Design and Performance of Earth Structure*, Geotechnical Special

Publication No. 25, ASCE. (edited by Lambe and Hansen), 613-628.

Tei, K., Taylor, R.N. & Milligan, G.W.E. (1998). "Centrifuge Model Tests of Nailed Soil Slopes." *Soils and Foundations*. Vol. 38, No. 2, June, Japanese Geotechnical Society, 165-177.

Wakins, A.T. & Powell, G.E. (1992). "Soil nailing to existing slopes as landslip preventive works." *Hong Kong Engineer*, March, 20-27.

ACKNOWLEDGEMENTS

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APPLYING IMAGE PROCESSING TECHNOLOGY TO BOULDER MAPPING

W.Z. Shi¹, K.C. Ng², C.Q. Huang³ and X.C. Li⁴

Abstract: Boulder fall is a potential hazard that affects urban development close to natural hillsides. Besides providing a time- and cost-effective alternative to the conventional method of boulder mapping, image processing technology improves the quality and objectiveness of the mapping results. Whilst application of image processing techniques to boulder mapping is subject to some technical constraints due to the complex natural setting, most of these could be overcome within the realm of current technologies. A detection accuracy of about 78% has been obtained for a trial study. By integrating digital photogrammetry, image analysis techniques and human-machine interaction, image analysis data can be used to compile statistics on boulder size, shape and height, which are valuable parameters for assessing boulder fall hazard.

INTRODUCTION

Urban development towards steep natural hillsides is subject to potential boulder fall hazard. Nine fatalities have occurred from rock/boulder falls between 1926 and 1995 (ERM 1995). Failure incidents involving boulders have been reported to the Geotechnical Engineering Office of the Civil Engineering Department every year, although no fatalities have been recorded in recent years.

Boulders on natural hillsides vary in a wide range of sizes and shapes, and the failure possibility of these boulders is difficult to predict. There is an established methodology for studying boulder fall hazard and engineering solutions (e.g. Au and Chan 1991; Chan et al. 2001). Boulder fall hazard studies generally require preparing a boulder distribution map, showing the location, size and shape of boulders, which forms the basis for design and implementation of in-situ stabilization and/or mitigation works.

Boulder mapping typically requires extensive fieldwork. The conventional method of boulder mapping is time-consuming, labour-intensive and costly, yet with questionable accuracy. Furthermore, the painstaking methods of mapping boulders on site, sometimes on steep and difficult terrain, may expose workers to unsafe field conditions.

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Because of the distinct physical features of boulders on ground surface, it is possible to extract information about boulder size, shape and height based on the application of image processing techniques on aerial photographs. Image processing techniques provide an alternative to the conventional method of boulder mapping and can greatly reduce the level of fieldwork required.

Ng et al. (2003) presented some preliminary results of a trial application of image processing techniques to boulder mapping and suggested that the techniques can improve the quality and objectiveness of producing boulder distribution map. As the follow-up study of the development described in Ng et al. (2003), this paper provides a new workflow of image analysis for boulder mapping and extraction of boulder properties. Furthermore, it proposes methods for enhancing the boulder mapping capability, including (a) enhancement of original image, and (b) binary processing with a different threshold for each area.

WORKFLOW OF IMAGE ANALYSIS FOR BOULDER MAPPING

The workflow of image analysis for boulder mapping is shown in Figure 1. The major processes involved are described in the following sections.

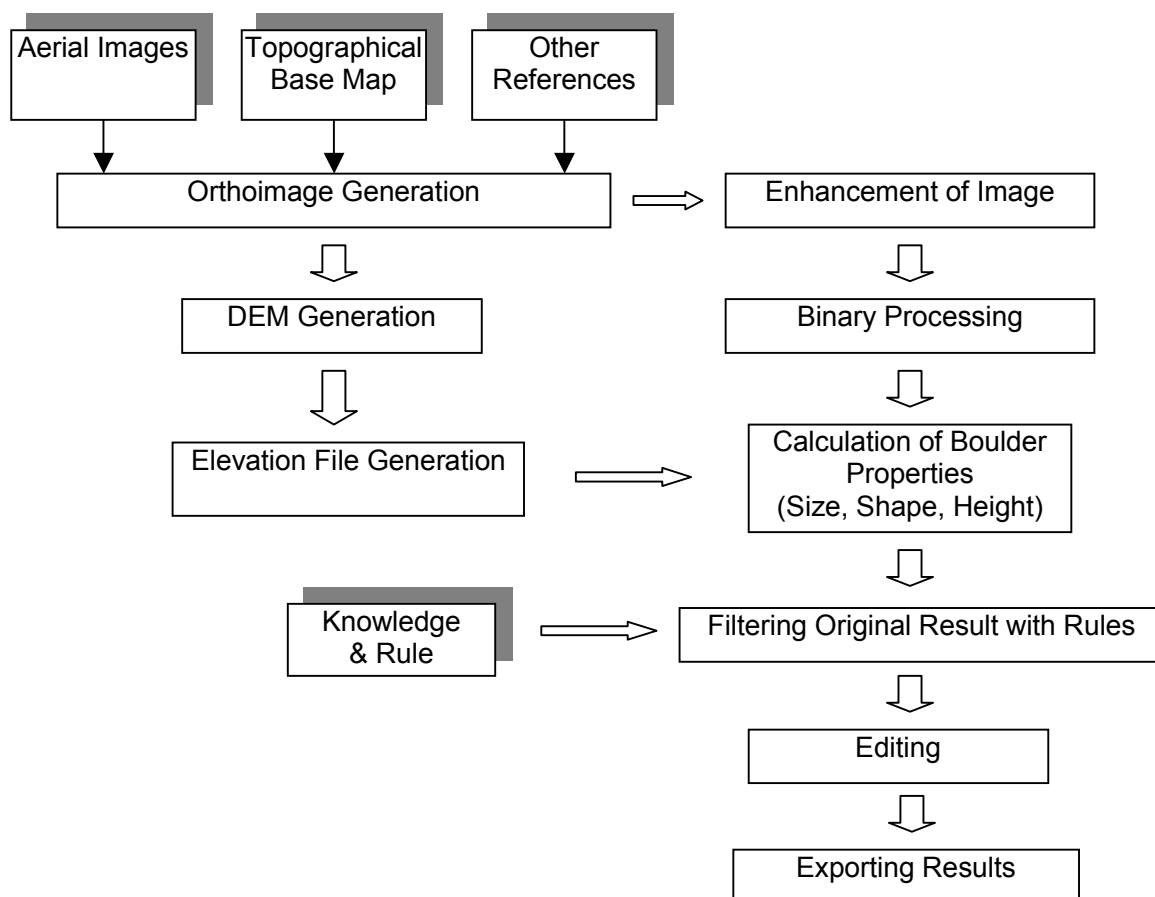


Figure 1 - Workflow of image analysis for boulder mapping.

Generation of Orthoimage

An efficient way for image processing is to divide an image into a set of small discrete rectangular grid cells, which are known as pixels. Each pixel is then given a value

representing its colour (Campbell 1987; Fisher et al. 1995; Jensen 1996). For detection of boulder and assessing boulder properties (e.g. size, shape), orthoimage instead of original aerial image is used. To create an orthoimage, two aerial photographs with at least 60% overlapped area are scanned as digital images and the relevant parameters and ground control points are selected from topographical base map or measured by carrying out fieldwork. Digital photogrammetric method is used to generate the orthoimage and the associated digital elevation model (DEM) for the area of interest.

Enhancement of Image

Image enhancement is applied to improve the visual appearance of an image for processing (Jensen 1996; Lillesand and Kiefer 2000). Image enhancement usually employs the techniques of (a) image contrasting to enhance the visibility of an object on the background, (b) image smoothing to reduce the noises from the image, and (c) image sharpening to enhance features, such as the outline of a structure.

Binary Processing

Binary processing changes a grayscale image into a binary image, to facilitate the later feature extraction processing (Colwell, 1983). In a binary image, the pixels have only two possible intensity values, which are normally displayed in black and white.

Calculation of Boulder Properties

Properties of a boulder, including size, shape and height, are useful for assessing boulder fall hazard. In binary images, the numbers and distribution of pixels reflect the size and shape of surface objects. The size and shape of each boulder can be derived from a continuous lattice of pixels with the same or similar value (Figure 2). The extraction of boulder height is based on the elevation file generated from the DEM of the area of interest.

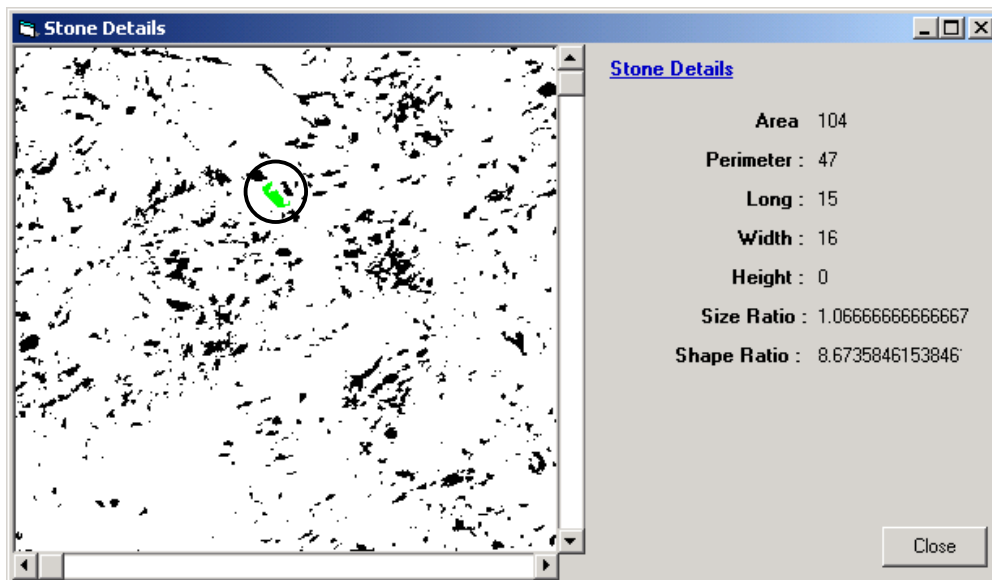


Figure 2 - Properties of a boulder (circled) extracted from the image analysis data.

Applying Filtering Rules

Through binary processing, it is possible to mis-interpret other ground features such as outcrops as boulders. Therefore, some filtering rules established by the selection of structural elements need to be applied. For example, as most outcrops are inevitably larger than boulders, a rule about boulder size can be set to remove most of the outcrops from the binary

results. Similarly, rules can be set about boulder shape to filter out outcrops from boulders. A proper filtering process based on size and shape can be used to delete unreasonable binary results so as to improve boulder accuracy.

Editing and Combining

According to the pre-set conditions (filtering rules), each selected boulder is presented as a polygon and a boulder distribution map can be generated for the study area. However, editing is still necessary to refine the results. The editing, which is a process of human-machine interaction, includes deletion of obvious errors from the detection results based on user's expert knowledge of the site conditions and integration of results obtained from applying different filtering rules and threshold values.

Exporting Results

The extracted boulder image (i.e. boulder distribution map) can be checked by overlaying it on the orthoimage or grayscale image (Figure 3a). To make it easy for user to assess boulder fall hazard, boulders can be graded with one or more properties and shown in different colours or gray tones (Figure 3b). The result of the image processing can be exported as a hard copy map, or as a soft image in various data formats.

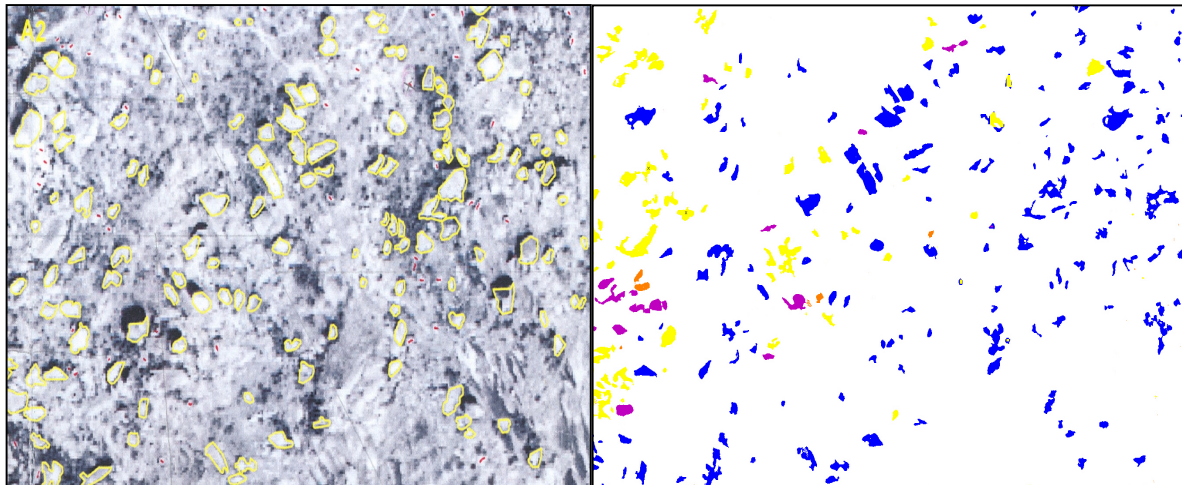


Figure 3 - Grading boulders and exporting results: a) overlaying boulders on the orthoimage, b) grading boulders of different properties.

TECHNIQUES FOR IMPROVING BOULDER DETECTION

On monochrome aerial photographs, different land cover and objects are distinguished based on their shapes and associated reflectance intensity. Boulders on natural hillsides that are bare or covered with very little vegetation would have relatively higher reflectance intensity than other land cover. Therefore, it is possible to define a threshold value to distinguish boulders from other land cover from aerial photographs. Those objects with reflectance intensity larger than the threshold can be classified as boulders.

Under ideal conditions, this method can achieve satisfactory results. However, in real world applications, there are many factors that could affect the identification result. For example, the brightness of boulders in different parts of an image may vary depending on where the boulders are located on the natural slopes and the aspect of the particular slopes. Furthermore, rock outcrops usually have similar reflectance intensity as boulders and they can

be wrongly classified as boulders, although their sizes are commonly much larger than boulders. Therefore, it is difficult to obtain high accuracy of boulder detection based on standard binary method only.

Other techniques would have to be incorporated to improve the accuracy of boulder detection. It is also necessary to utilize expert's knowledge about characteristics of the boulders and local setting of the area of interest, during image processing and boulder detection process. The following techniques have been applied to further improve the accuracy of boulder detection:

- (a) To enhance the image based on some newly developed algorithms, such as linear enhancement and local spatial enhancement.
- (b) To apply different thresholds to different parts of the image in binary processing, based on a pre-partitioning of the whole image.
- (c) To further edit and combine the automatically extracted results by incorporating expert's knowledge.

Enhancement of Original Image

In order to make the image more interpretable and to improve the accuracy of boulder detection, it is necessary to enhance the image before detection. Two image enhancement techniques are proposed, namely linear enhancement and local spatial enhancement.

When the contrast of an image is low, reflectance intensity of most pixels are resided in a narrow range and it is hard for user to interpret and set appropriate binary threshold. Therefore, linear enhancement is performed to enlarge the range of reflectance intensity distribution so as to enhance the image contrast. Figure 4 illustrates the results of the enhancement process. The original image is blurry and it is hard to distinguish boulders from the background (Figure 4a). After applying linear enhancement, the image is brighter and becomes easier to distinguish boulders from the background (Figure 4b).

Local spatial enhancement can make boulder brighter and the background darker. After applying local spatial enhancement to an image, it is not only easier to distinguish boulders from the background (Figure 4c), but also possible to select a suitable threshold to reduce both omission and commission errors.

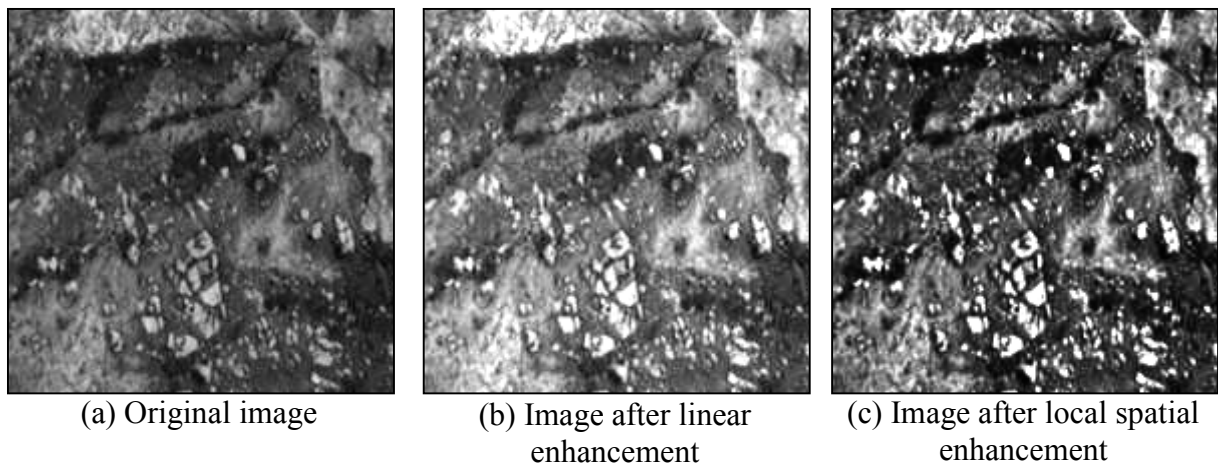


Figure 4 - Image enhancement techniques.

Applying Different Thresholds in Binary Processing

In binary processing, a threshold value is first specified. All pixels with intensity higher than the threshold will be classified as boulder pixels. The selected threshold has a direct and important effect on boulder detection accuracy. In a real-life application, a suitable selection of threshold should be based on the knowledge of the area of interest from fieldwork and expert experience about the site condition. However, it is sometimes hard to select one threshold that is suitable for the whole image.

Figure 5 illustrates the results of boulder detection by applying different threshold values for different parts of the image. Using a high threshold, most boulders in the upper part of the image have been correctly detected, but most boulders in the lower part are omitted (Figure 5b). In contrast, when a low threshold is applied, most background pixels in the upper part of the image are classified as boulders, although boulders in the lower part have been correctly detected (Figure 5c). By applying different thresholds to two areas of the image, most boulders in both parts have been recognized (Figure 5d).

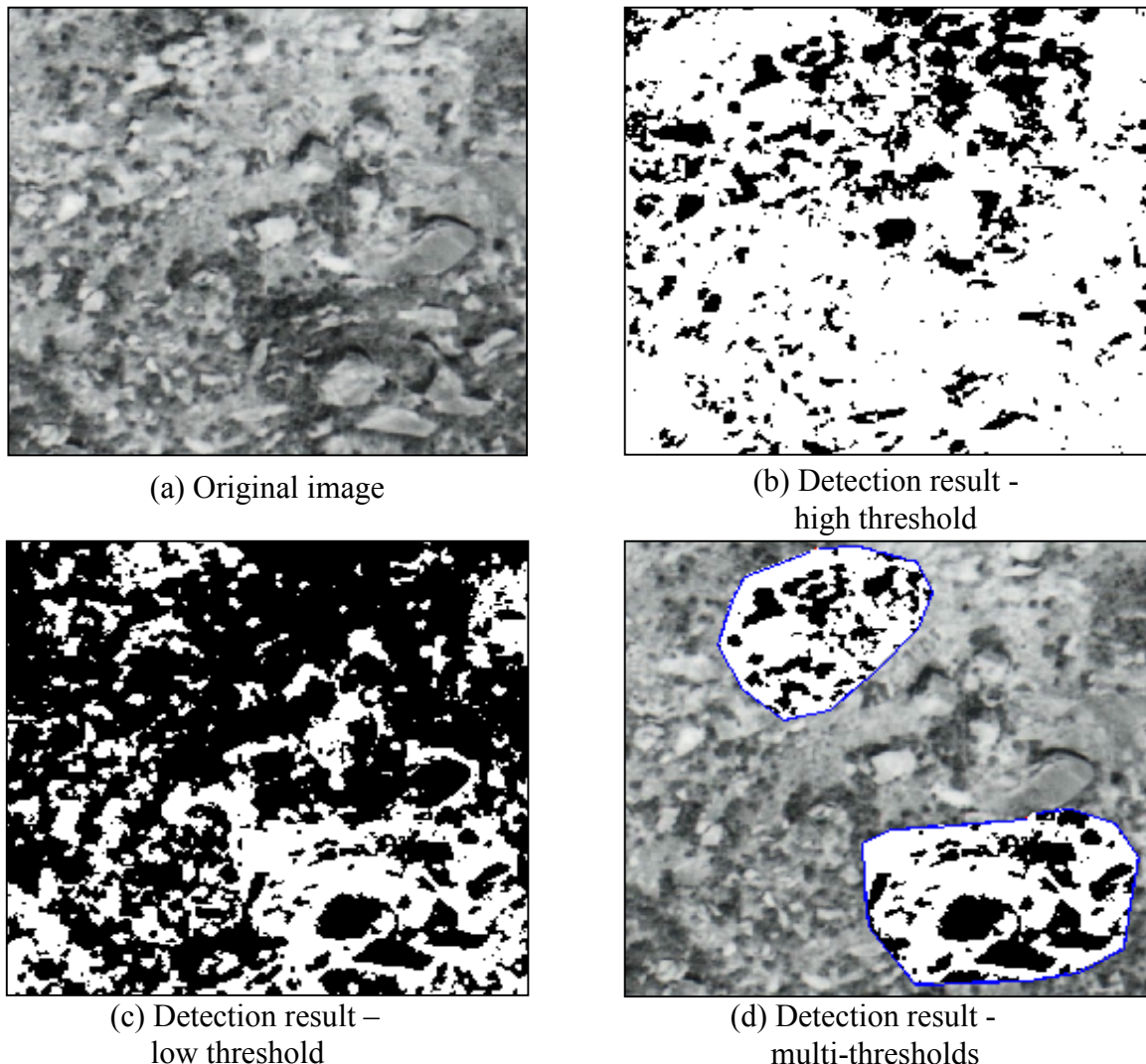


Figure 5 - Boulder detection by applying different thresholds.

TRIAL STUDY AND RESULTS

In order to test the feasibility and accuracy of the methods presented above, a trial study was conducted for a site in Chung Hom Kok, Hong Kong.

Data

Three monochrome aerial photographs (Figure 6; Serial No. 6679, 6680 and 6681) taken on 1 February 1963 were used in the study. The test site is in the overlapped area of the three aerial photographs. Because only paper prints were available, a desktop scanner was used to scan the photographs into digital images in 1200 dpi.

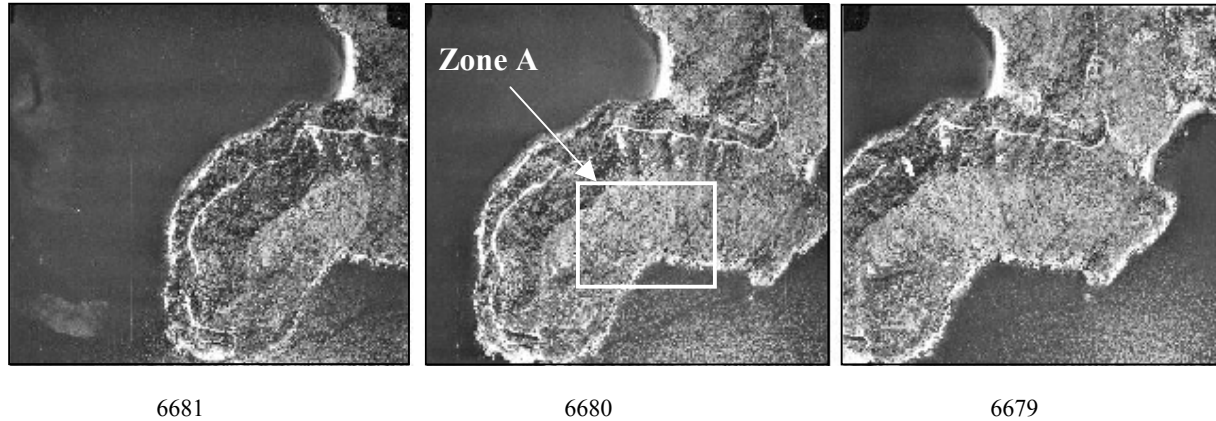


Figure 6 - Monochrome aerial photographs of the trial study area.

For generating the DEM, a 1:600-scale topographical base map created in 1960's was used to obtain the ground information near the time of the aerial photography in 1963. Fieldwork was carried out to measure boulder heights (Table 1) for checking purposes.

Table 1 - Height and coordination of test boulders

Boulder No.	Easting	Northing	Height (m) from Field Measurement	Height (m) from Photogrammetry
R7	838881	808358	1.06	1.9
R1	838862	808352	2.18	2.7
R2	838846	808377	3.46	3.8
R6	838835	808390	3.02	4.2
R5	838839	808407	3.81	4.2
R8	838847	808426	2.28	3.3
B11	838825	808436	2.00	2.2
B12	838832	808452	2.59	3.5
B13	838833	808456	2.86	2.7

Detection Result

Taking zone A of Figure 6 as an example, the overall boulder detection accuracy based on the proposed new solutions is 77.8% (Table 2).

Table 2 - Detection accuracy of boulder in Zone A based on the proposed solutions

Actual number of boulders	Detected boulders	Neglected boulders	Test Accuracy %
185	144	41	77.8

Boulder Properties

The size and shape of the boulders detected are in good agreement with those seen on the aerial photographs. These image data can be used for classifying the boulders in terms of

boulder density, and size and shape distribution. Comparing the height of boulders measured from fieldwork with those obtained from digital photogrammetry, it shows the variation ranges from several centimetres to 1.5 m (Table 1). This relatively large range of deviation is due to the resolution of the DEM generated by the digital photogrammetric techniques. Nevertheless, the overall boulder properties extracted are of acceptable quality for use in boulder fall hazard assessment.

CONCLUSIONS

An improved approach and workflow for extracting boulders based on image processing technology was developed, integrating digital photogrammetry, image analysis techniques and human-machine interaction. The boulder detection accuracy has been greatly improved by applying several techniques, including linear and local spatial image enhancement, a different threshold for each participated image, and human-machine interaction. A trial experiment at a site in Chung Hom Kok gave a detection accuracy of about 78% and the boulder properties extracted are of acceptable quality for assessing boulder fall hazard.

REFERENCES

- Au, S.W.C., and Chan, C.F. (1991). "Boulder treatment in Hong Kong." *Selected Topics in Geotechnical Engineering*, Li, K.S., Editor, Lumb Volume, 39-71.
- Campbell, J.B. (1987). *Introduction to Remote Sensing*. The Guilford Press, New York-London.
- Chan, Y.C., Chan, C.F., and Au, S.W.C. (2001). "Design of boulder fence in Hong Kong." *The Practice of Geotechnical Engineering in Hong Kong*, Li, K.S. and Lo, S-C.R., Editors, Centre for Research & Professional Development, Hong Kong, pp. 137-150.
- Colwell, R.N. (1983). *Manual of Remote Sensing (Second Edition)*. American Society of Photogrammetry, Falls Church, Virginia.
- ERM-Hong Kong Ltd. (1995). *Feasibility Study for QRA of Boulder Fall Hazard in Hong Kong*, GEO Report No. 80, Geotechnical Engineering Office, Hong Kong.
- Fisher, B., Perkins, S., Walker, A., and Wolfart, E. (1995). *Hypomedical Image Processing Reference*. University of Edinburgh, UK, <<http://www.cce.hw.ac.uk/hipr/html/glossary.html>>.
- Jensen, J.R. (1996). *Introductory Digital Image Processing*. 2nd Edition, Prentice Hall, New Jersey.
- Lillesand, T.M., and Kiefer, R.W. (2000). *Remote Sensing and Image Interpretation*. 4th Edition, John Wiley & Sons, Inc., New York.
- Ng, K.C., Li, X.C., Shi, W.Z., Zhu, C.Q., and Shum, W.L. (2003). "Detection of boulders on natural terrain using image processing techniques." *Proceedings of Intelligent Engineering Application of Digital Remote Sensing Technology*, Hong Kong Institution of Engineers, 11 April 2003, Hong Kong, 55-63.

ACKNOWLEDGEMENTS

The work was a joint project of the Geotechnical Engineering Office and the Department of Land Surveying and Geo-informatics, Hong Kong Polytechnic University. The research development of this project is supported by CRC program RGC Hong Kong (Project No.: 3_ZB40) and The Hong Kong Polytechnic University (Project No.: 9709). This paper is published with the permission of the Head of the Geotechnical Engineering Office and the Director of Civil Engineering of the Government of the Hong Kong Special Administrative Region.

PRESCRIPTIVE SOIL NAIL DESIGN FOR CONCRETE AND MASONRY RETAINING WALLS

B. L. S. Lui¹ and Y. K. Shiu¹

Abstract: Use of soil nails as prescriptive measures for upgrading soil cut slopes have been successfully applied since 1996. As an extension to the scope of such measures, new prescriptive soil nail design has also been developed for improving stability of substandard concrete or masonry retaining walls. The development of the new prescriptive design involved a detailed review of a number of past design cases that had used analytical design of soil nails for upgrading retaining walls. In the review, statistical relationship between lengths of soil nails used and heights of the wall features was established and examined. This paper describes the procedure and approach adopted for developing the new prescriptive soil nail design. It also presents qualifying criteria for the application of the prescriptive design, along with guidelines for using the design. Recommended design and construction considerations for the prescriptive design are also provided.

INTRODUCTION

Since the publication of the first edition of GEO Report No. 56 "Application of Prescriptive Measures to Soil Cut Slopes" (Wong and Pang, 1996) in October 1996, use of soil nails as prescriptive measures for upgrading soil cut slopes have been successfully applied in Hong Kong. Prescriptive Measures are pre-determined, experience-based and suitably conservative modules of works prescribed to a slope or retaining wall to improve its stability or reduce the risk of failure, without detailed ground investigation and design analyses (Wong et al, 1999). There has also been continuous development to improve guidelines on and extend the scope of application of prescriptive measures. As a new initiative to extend the scope of such measures, prescriptive soil nail design has recently been developed for improving stability of substandard concrete or masonry retaining walls.

DERIVATION OF PRESCRIPTIVE SOIL NAIL DESIGN GUIDELINES FOR CONCRETE/MASONRY RETAINING WALLS

Guidelines for prescriptive design of soil nails for soil cut slopes are given in MMBP & VLA (1999). They form the framework for the derivation of prescriptive soil nail designs for upgrading concrete/masonry retaining walls. As such, the basis of the prescriptive soil nails for soil cut slopes is outlined first for ease of reference. The prescriptive soil nail layout for soil cut slopes is based on 469 data sets of soil nailed slopes. The design prescriptions distinguish between the required range of increase in factor of safety (ΔFOS) for the slope to be upgraded and the maximum effective height of the slope (H_e).

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The maximum effective height H_e is given by the following equation (Wong & Ho, 1995):

$$H_e = H * (1 + 0.35 \tan \beta) + q/20$$

where H = Maximum height from the toe to the crest of the slope feature (m)
 β = Average gradient of the ground above the crest of the slope feature
 q = Surcharge loading expressed as equivalent uniform pressure (kPa)

Three ranges of ΔFOS to be achieved by the prescriptive soil nails are categorized below:

- Range A for a large ΔFOS ($0.3 < \Delta FOS \leq 0.5$)
- Range B for a medium ΔFOS ($0.1 < \Delta FOS \leq 0.3$)
- Range C for a small ΔFOS ($0 < \Delta FOS \leq 0.1$)

For each range of ΔFOS , the prescriptive soil nail design was developed from the data sets of soil nailed slopes by following the steps below:

- (i) Prepared a graph of total nail length versus maximum effective slope height, and plotted an upper bound envelope for total nail length in the graph.
- (ii) Prepared a graph of soil nail length versus maximum effective slope height, and plotted a design envelope for the soil nail lengths in the graph. The envelope was based on the 90th percentile values of the data sets.
- (iii) Determined the nail density required for different slope heights from the two design envelopes.

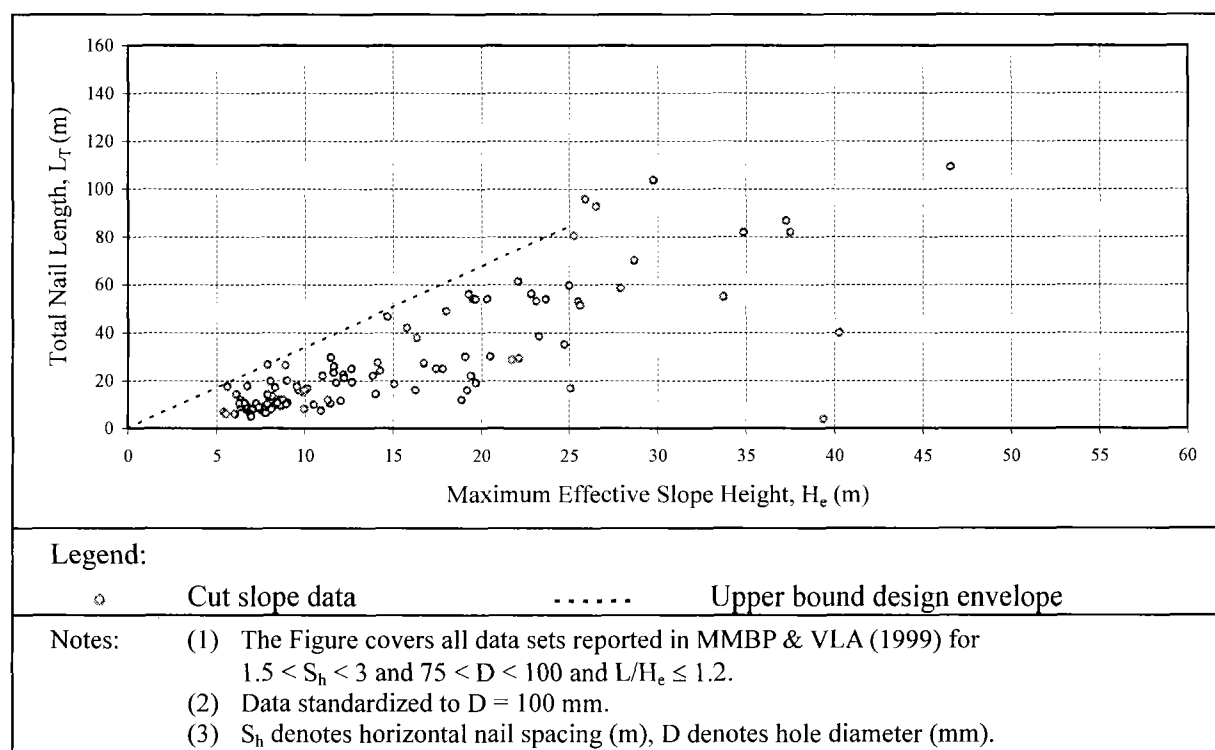


Figure 1 - Total Nail Length versus Maximum Effective Slope Height for Cut Slopes in Range A, i.e. $0.3 \leq \Delta FOS < 0.5$

For slopes in Range A, Figures 1 and 2 show the upper bound design envelopes in terms of effective slope height plotted against the total nail length and individual nail length respectively. Similar pair of plots for the slopes in each of ΔFOS Ranges B and C were also produced. These envelopes form the basis of the prescriptive design guidelines for concrete and masonry retaining walls.

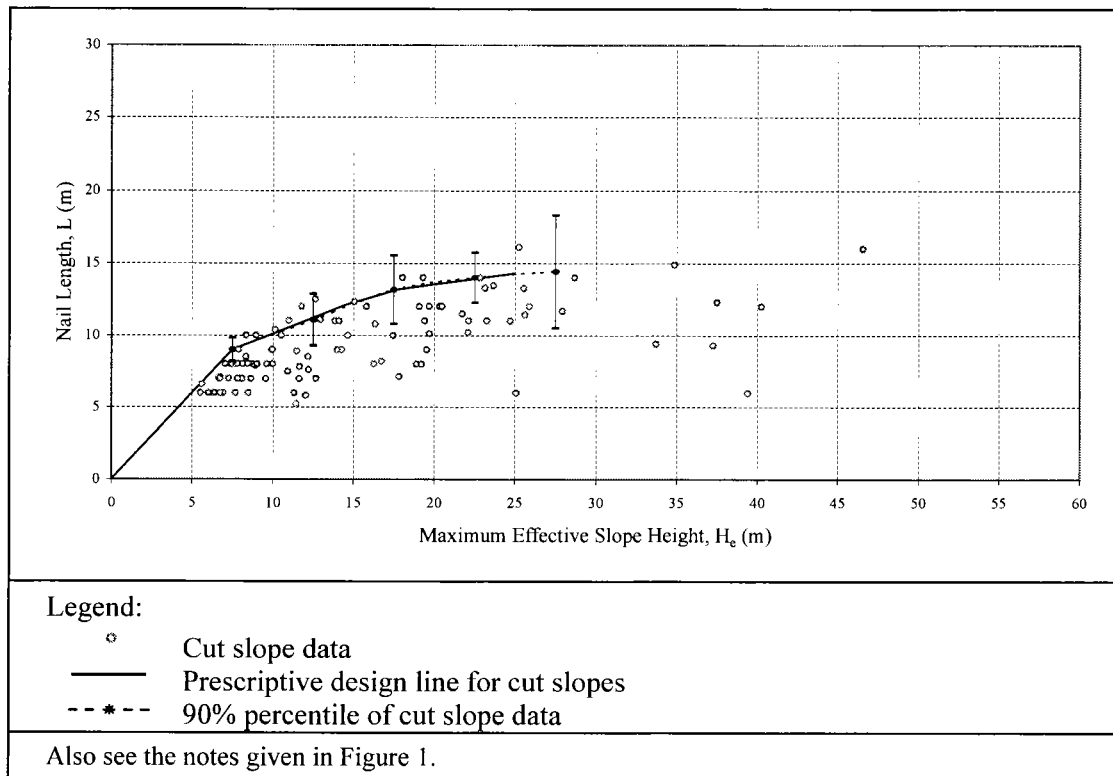


Figure 2 - Nail Length versus Maximum Effective Slope Height for Cut Slopes in Range A, i.e. $0.3 \leq \Delta FOS < 0.5$

Data Used for Deriving the Prescriptive Design Guidelines for Concrete/Masonry Retaining Walls

Designs of 80 CR and R features which were upgraded by soil nails have been reviewed in this study. The CR features are cut slopes with a retaining wall at the toe of the slope, whereas R features are retaining walls supporting a slope or a platform with a surface sloping at $<15^\circ$ (fill or natural ground). As more than one cross-section have been analysed in a few features, a total number of 113 data sets have been studied.

In the review, 71 data sets coming from 51 features were found with exceptional situations that rendered these cases not suitable for use in deriving the prescriptive design line. Forty-two data sets coming from 29 features reviewed were found suitable for use in deriving the prescriptive design guidelines.

For the purpose of deriving the prescriptive soil nail design for CR or R features involving concrete/masonry retaining walls, the total length of a soil nail, L , is considered to be composed of the following components:

$$L = L_{\text{free}} + L_s$$

where L_{free} = Length of the portion of soil nail within retaining wall and fill layer (m)
 L_s = Length of the portion of soil nail in material behind the fill and retaining wall (m)

Derivation of Prescriptive Soil Nail Design for CR Features

It is considered that soil nailed CR features behave like nailed cut slopes. In the past design cases reviewed, the nailed CR features were all designed in the same way as that for nailed cut slopes. Hence the three ranges of ΔFOS for prescriptive nailed soil cut slopes can also be applied to nailed CR features.

Using the data sets of CR features, the following plots were prepared for each of the three ΔFOS ranges:

- (i) total soil nail length in material behind the fill and retaining wall per metre run, $L_{s(\text{Total})}$, versus the effective feature height, and
- (ii) average length of the portion of soil nail in material behind the fill and retaining wall, $L_{s(\text{Average})}$, versus the effective feature height.

In these plots, the lengths of the portion of soil nails in the retaining wall and fill of the CR features, L_{free} , were excluded. The plots form the basis for formulating the prescriptive design guidelines.

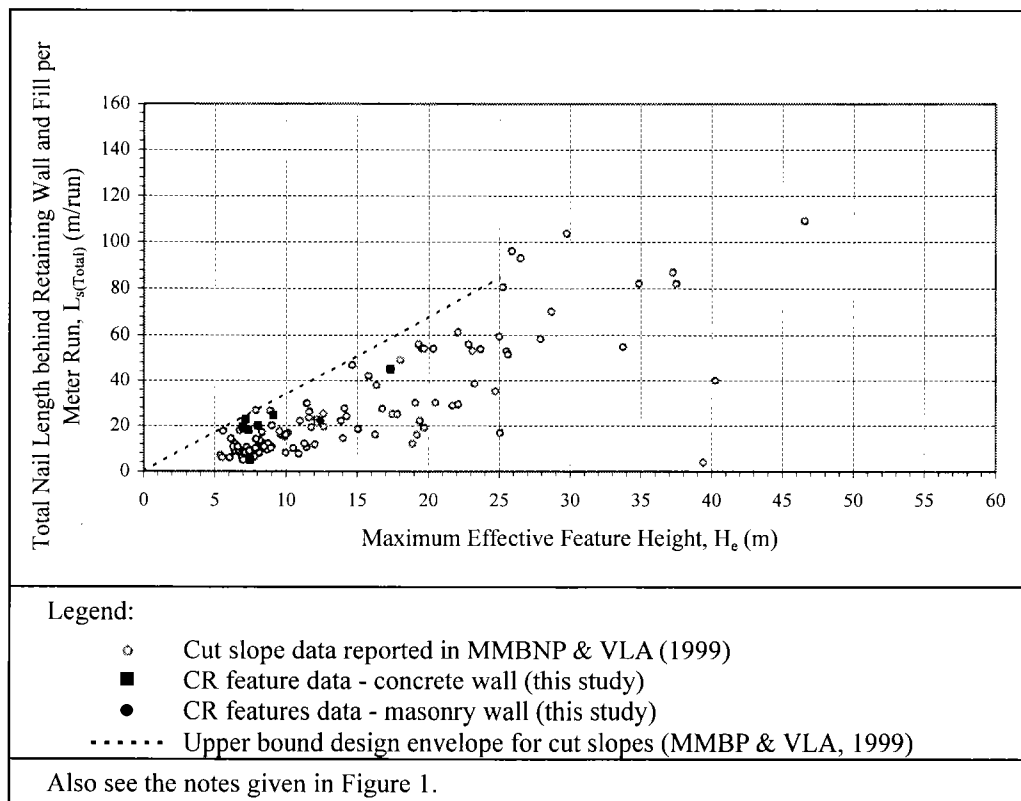


Figure 3 - Total Nail Length versus Maximum Effective Feature Height for CR Features and Cut Slopes in Range A, i.e. $0.3 \leq \Delta\text{FOS} < 0.5$

Eight designs of the features are within Range A. Figures 3 and 4 show the plots of $L_s(\text{Total})$ and $L_s(\text{Average})$ against maximum effective feature heights respectively. The length of the portion of the soil nail in retaining wall and fill behind, i.e. L_{free} , of the data sets has been discarded. For comparison purpose, the data on nailed soil cut slopes and the prescriptive design lines for total nail length and nail length of soil cut slopes are also plotted in the figures.

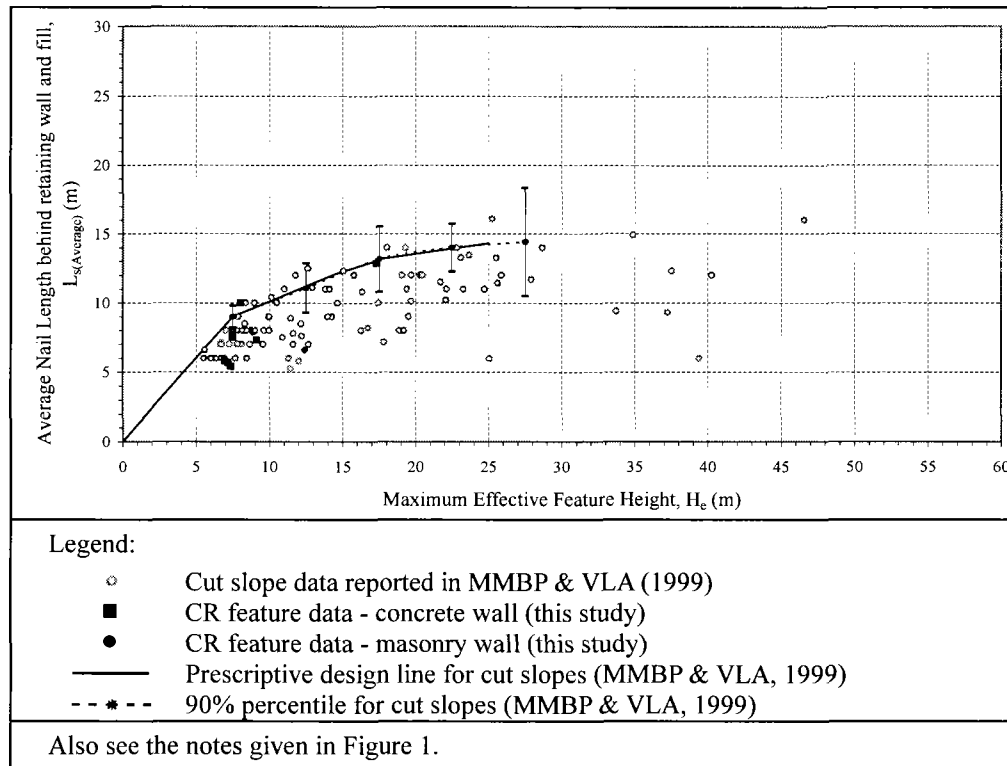


Figure 4 - Average Nail Length versus Maximum Effective Feature Height for CR Features and Cut Slopes in Range A, i.e. $0.3 \leq \Delta F < 0.5$

Similarly, $L_s(\text{Total})$ and $L_s(\text{Average})$ plots are also produced for each of Ranges B and C. There are 13 and 6 designs of the features within Ranges B and C respectively.

In the plots of $L_s(\text{Total})$, all data sets of the CR features are below the prescriptive design lines of total nail length derived for soil cut slopes for the three ranges of ΔFOS . For the plots of $L_s(\text{Average})$, all the data sets of the CR features, except a few, are below the prescriptive design lines of nail length derived for soil cut slopes. The few data points above the design lines exceed marginally the prescriptive design values (See Figure 4 for example). Since the prescriptive design line of nail length for soil cut slopes is based on the 90th percentile values and the data point exceeds the 90th percentile value by only a very small amount, the data point is considered suitable for use. The $L_s(\text{Total})$ and $L_s(\text{Average})$ plots illustrate that the prescriptive soil nail design for soil cut slopes is also applicable to CR features with concrete/masonry retaining walls. That means L_s for CR features would be the same as soil nail lengths for soil cut slopes.

Table 1 gives the standard prescriptive soil nail layout and L_s for CR features with a toe wall. For a complete prescriptive design, it is necessary to determine the free length (L_{free}) of the soil nails.

Table 1 - Standard Prescriptive Soil Nail Layout for CR Features with a Toe Wall (After Lui & Shiu, 2004)

Standard Soil Nail Layout	H_e (m)	ϕ_r (mm)	ϕ_h (mm)	A			B			C		
				N	L_s (m)	S_h (m)	N	L_s (m)	S_h (m)	N	L_s (m)	S_h (m)
(a)	3	25	100	2	4	1.5	2	4	1.5	2	4	1.5
(b)	4	25	100	2	5	1.5	2	5	1.5	2	5	1.5
(c)	5	25	100	3	6	1.5	3	6	1.5	3	6	2.0
(d)	6	25	100	4	8	1.5	3	8	1.5	3	7	1.5
(e)	7	25	100	4	9	1.5	4	8	1.5	3	7	1.5
(f)	8	25	100	5	9	1.5	4	8	1.5	3	8	1.5
(g)	9	25	100	5	10	1.5	4	9	1.5	4	8	1.5
(h)	10	25	100	6	10	1.5	4	10	1.5	4	9	1.5
(i)	12	32	100	6	11	1.5	5	10	1.5	5	10	2.0
(j)	14	32	100	6	12	1.5	5	11	1.5	6	10	2.0
(k)	16	32	100	7	12	1.5	7	12	2.0	6	11	2.0
(l)	18	32	100	8	13	1.5	8	12	2.0	7	12	2.0
(m)	20	32	100	10	14	2.0	9	12	2.0	8	12	2.0
(n)	22	32	100	11	14	2.0	10	12	2.0	8	12	2.0
(o)	24	32	100	12	14	2.0	10	12	2.0	8	12	2.0
(p)	25	32	100	12	15	2.0	10	12	2.0	8	12	2.0

Notes: (1) 'A', 'B' and 'C' refer to factor of safety increase in the ranges 0.3 to 0.5, 0.1 to less than 0.3, and less than 0.1 respectively.
(2) L_s is the length of the portion of soil nail in material behind the retaining wall and fill.
(3) Total length of a soil nail should include the length of the portion of nail within the retaining wall and fill, L_{free} , and L_s .
(4) H_e is the maximum effective feature height, ϕ_r the steel bar diameter, ϕ_h the hole diameter, N the number of rows of soil nails and S_h the horizontal spacings of soil nails.
(5) Soil nails should be evenly spaced over the feature face.

Free Length of Soil Nails (L_{free}) for CR Features

Five data sets of CR features used for deriving the prescriptive design guidelines are found to have fill behind the retaining wall at toe. Figure 5 shows a plot of free length (L_{free}) against retaining height of the wall (H_r) for the five data sets. It shows that 2 m is the upper bound of L_{free} for all the CR features with concrete/masonry retaining wall. Therefore in the absence of information on wall thickness and extent of fill behind the wall, L_{free} of 2 m may be assumed for the prescriptive design for CR features.

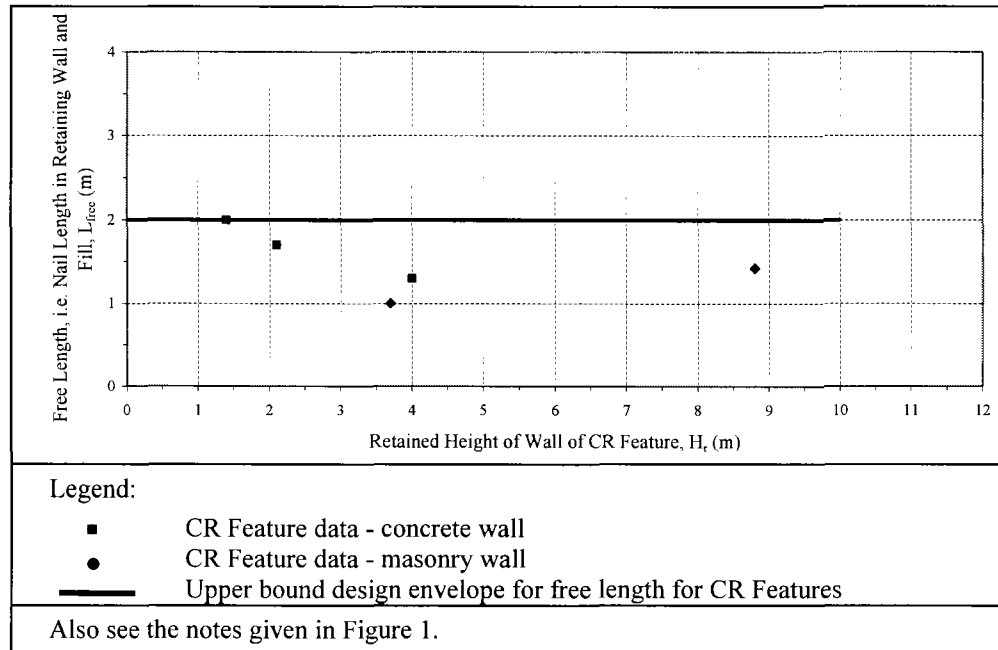


Figure 5 - Free Length versus Retained Height for CR Features

Derivation of Prescriptive Soil Nail Design for R Features

In those past design cases reviewed that involved upgrading of R features by soil nails, designers normally carried out checks to ensure that the minimum FOS against overturning and sliding of the retaining wall were achieved. The whole nailed R feature can be regarded as a nailed cut slope with a steep facing. In this respect, the prescriptive soil nail design for soil cut slopes would also be applicable to R features with concrete/masonry retaining walls. For a masonry wall with slenderness ratio greater than 5 or with dry packed masonry, the wall may not have adequate structural integrity to behave as a single body with the soil nails and the soil mass behind. Hence, a reinforced concrete skin wall should be constructed and it should be attached to the soil nails at the front face of the masonry wall.

Two wall standards, i.e. new and existing wall standards (GCO, 1984), have been considered. To develop the prescriptive design guidelines for the R features, $L_{s(Total)}$ and $L_{s(Average)}$ of the R features are plotted against effective feature height respectively for each of the two wall standards. The following sections discuss the results obtained.

New Wall Standard

Five designs of features are found to have been upgraded using the new wall standard. The data sets for these features are plotted in Figures 6 and 7 for $L_{s(Total)}$ and $L_{s(Average)}$ respectively. L_{free} of the data sets have been discarded. The prescriptive design lines for soil cut slopes in Range A are also plotted in the two figures for comparison purpose.

Existing Wall Standard

Ten designs of features are found to have been upgraded using the existing wall standard. The data sets for these features are plotted in Figures 8 and 9 for $L_{s(Total)}$ and $L_{s(Average)}$ respectively. L_{free} of the data sets have been discarded. The prescriptive design lines for soil cut slopes in Range B are also plotted in the two figures for comparison purpose.

Discussion

As shown in Figures 6 and 7, all the data sets based on the new wall standard are below the prescriptive design lines derived for soil cut slopes in Range A. It is therefore considered adequately safe to adopt the prescriptive soil nail lengths for CR features in Range A as given in Table 1 as L_s for upgrading R features with concrete/masonry retaining walls using the new wall standard.

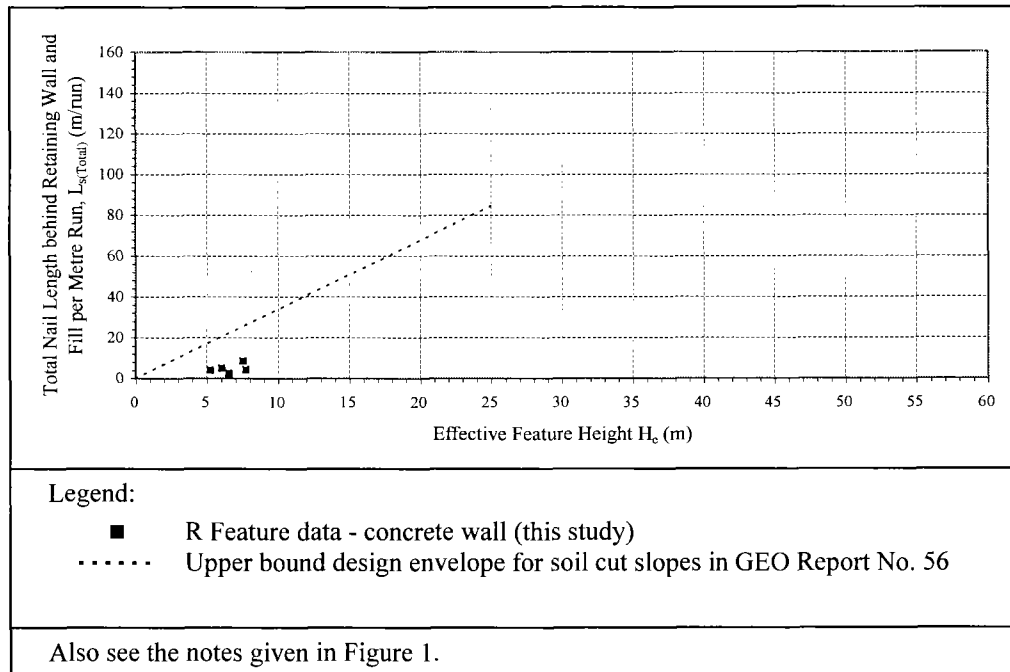


Figure 6 - Total Nail Length versus Maximum Effective Feature Height for R Features with New Wall Standard

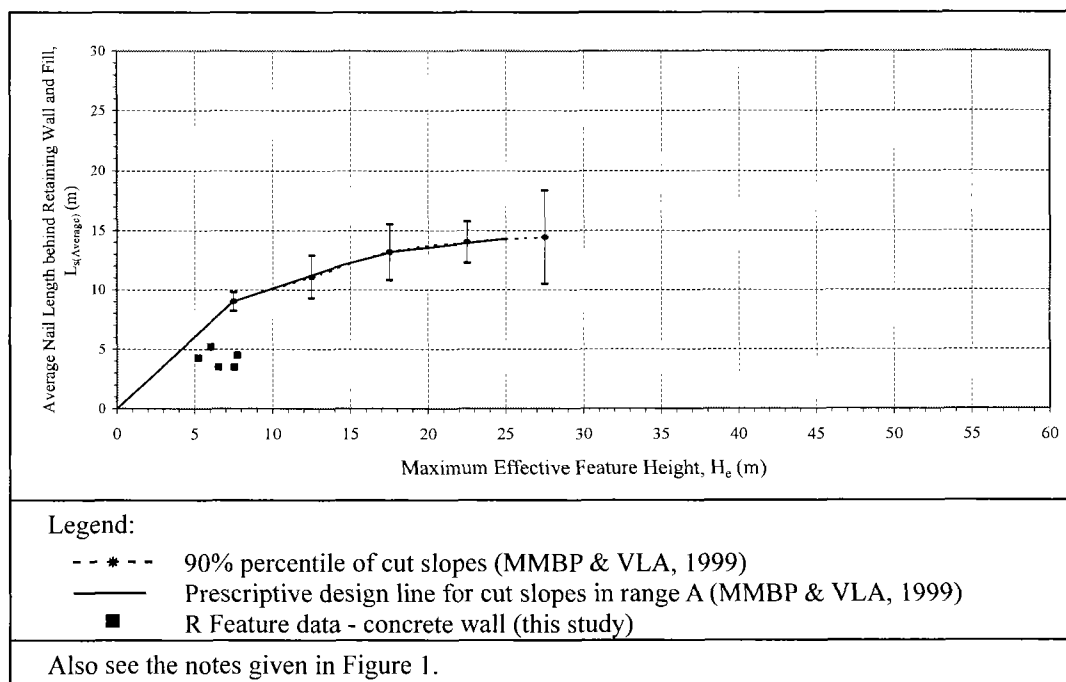


Figure 7 - Average Nail Length versus Maximum Effective Feature Height for R Features with New Wall Standard

Figures 8 and 9 indicate that the data sets based on the existing wall standard are below the prescriptive design lines derived for soil cut slopes in Range B. It is reasonable to adopt the prescriptive soil nail lengths for CR features in Range B as given in Table 1 as L_s for upgrading R features with concrete/masonry retaining walls using the existing wall standard.

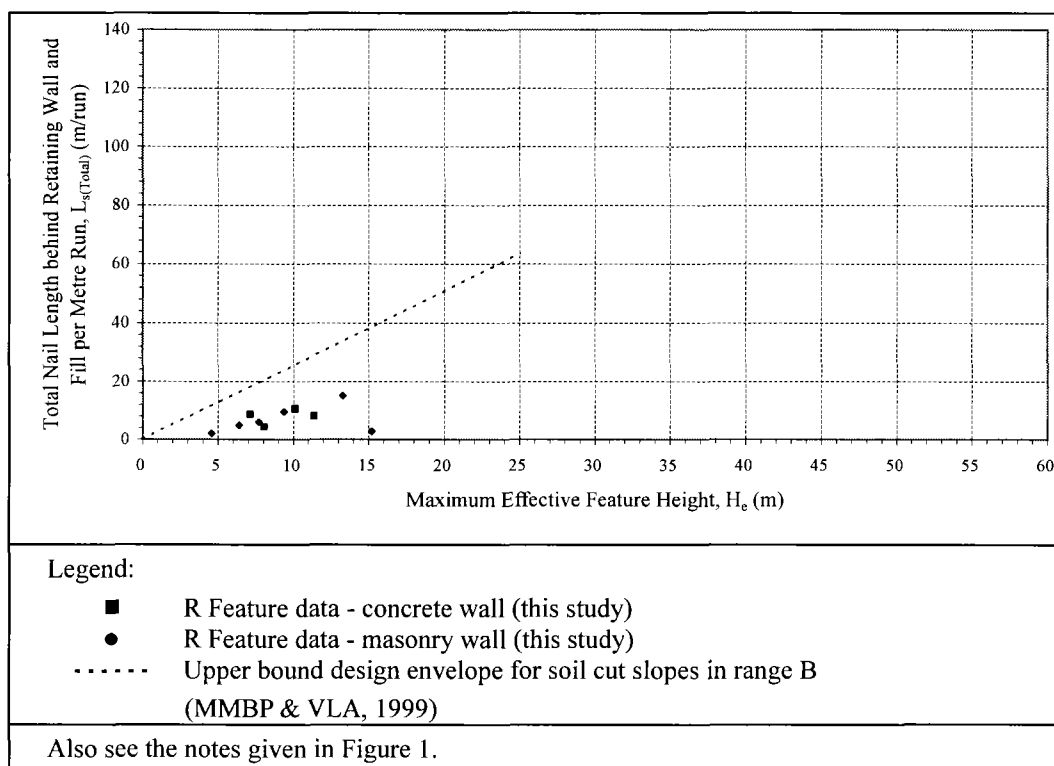


Figure 8 - Total Nail Length versus Maximum Effective Feature Height for R Features with Existing Wall Standard

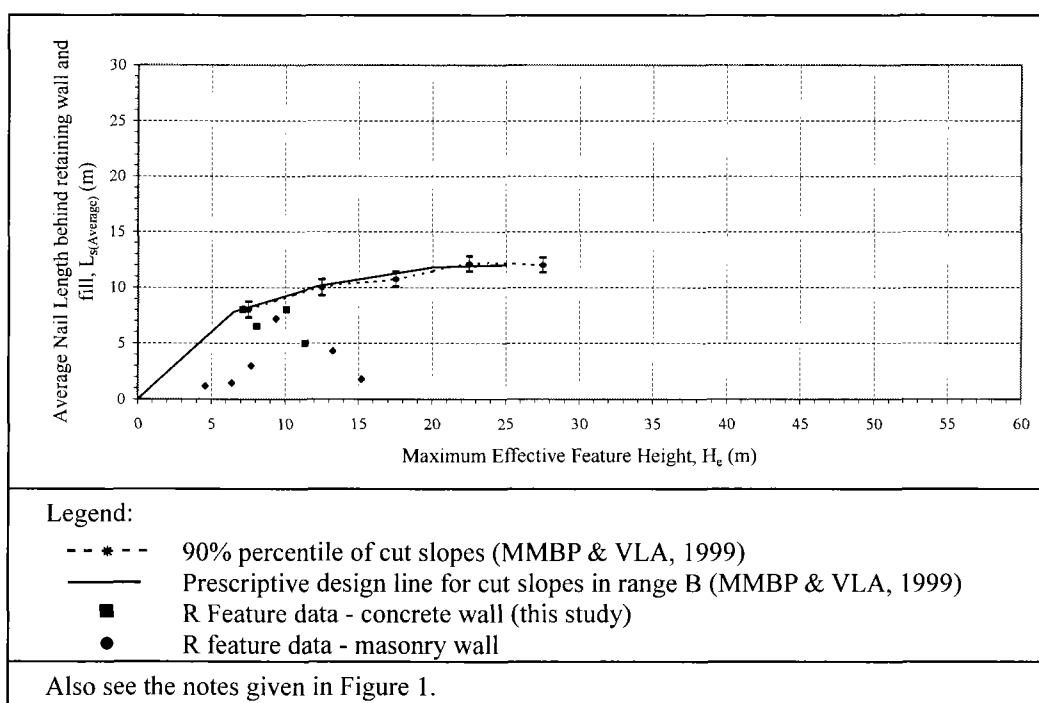


Figure 9 - Average Nail Length versus Maximum Effective Feature Height for R Features with Existing Wall Standard

Table 2 gives the standard soil nail layout and L_s for both new wall and existing wall standard. For a complete design, it is necessary to determine L_{free} .

Table 2 - Standard Prescriptive Soil Nail Layout for R Features of Concrete/Masonry Retaining Wall (after Lui & Shiu, 2004)

Standard Soil Nail Layout	H_e (m)	ϕ_r (mm)	ϕ_h (mm)	New Wall Standard			Existing Wall Standard		
				N	L_s (m)	S_h (m)	N	L_s (m)	S_h (m)
(a)	3	25	100	2	4	1.5	2	4	1.5
(b)	4	25	100	2	5	1.5	2	5	1.5
(c)	5	25	100	3	6	1.5	3	6	1.5
(d)	6	25	100	4	8	1.5	3	8	1.5
(e)	7	25	100	4	9	1.5	4	8	1.5
(f)	8	25	100	5	9	1.5	4	8	1.5
(g)	9	25	100	5	10	1.5	4	9	1.5
(h)	10	25	100	6	10	1.5	4	10	1.5

Notes: (1) New and existing wall standards refer to the standards of upgrading retaining wall in accordance with the conditions listed in the Geotechnical Manual for Slopes (GCO, 1984).
(2) Refer to Notes (2) to (5) given in Table 1.

Free Length of Soil Nails (L_{free}) for R Features

Figure 10 shows a plot of L_{free} against H_r of R features which have fill behind the walls. The data points are below an upper bound envelope set at $L_{free} = H_r$. Therefore in the absence of information on wall thickness and extent of fill behind the wall, designers may assume $L_{free} = H_r$. The L_{free} derived for R features is larger than that for CR features. This is reasonable since the fill behind a R feature is expected to be in a larger extent than in CR features.

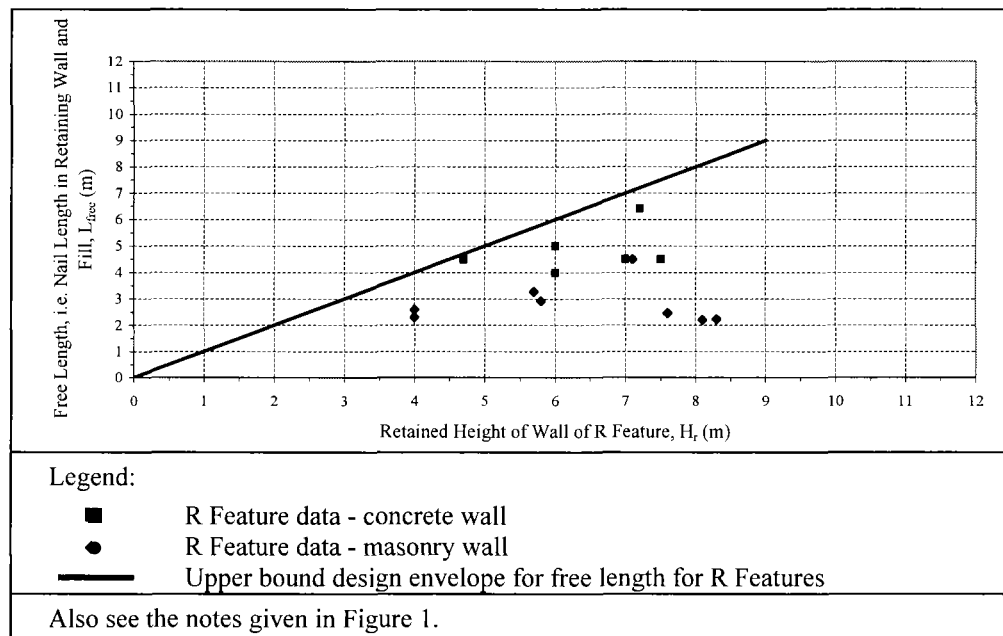


Figure 10 - Free Length versus Retained Height for R Features

SCOPE OF APPLICATION AND DESIGN PROCEDURES

The scope and qualifying criteria of the application of prescriptive soil nails to CR and R features are given in Lui and Shiu (2004). The qualifying criteria are mainly related to height limits, past performance and hydrogeological conditions of the features. The prescriptive soil nails can be applied as upgrading works if the criteria are satisfied. Where the qualifying criteria are not satisfied, the prescriptive soil nails can still be applied as preventive maintenance.

Detailed design procedures are also provided in Lui and Shiu (2004). Figures 11 and 12 show the typical patterns of soil nailing for CR and R features respectively.

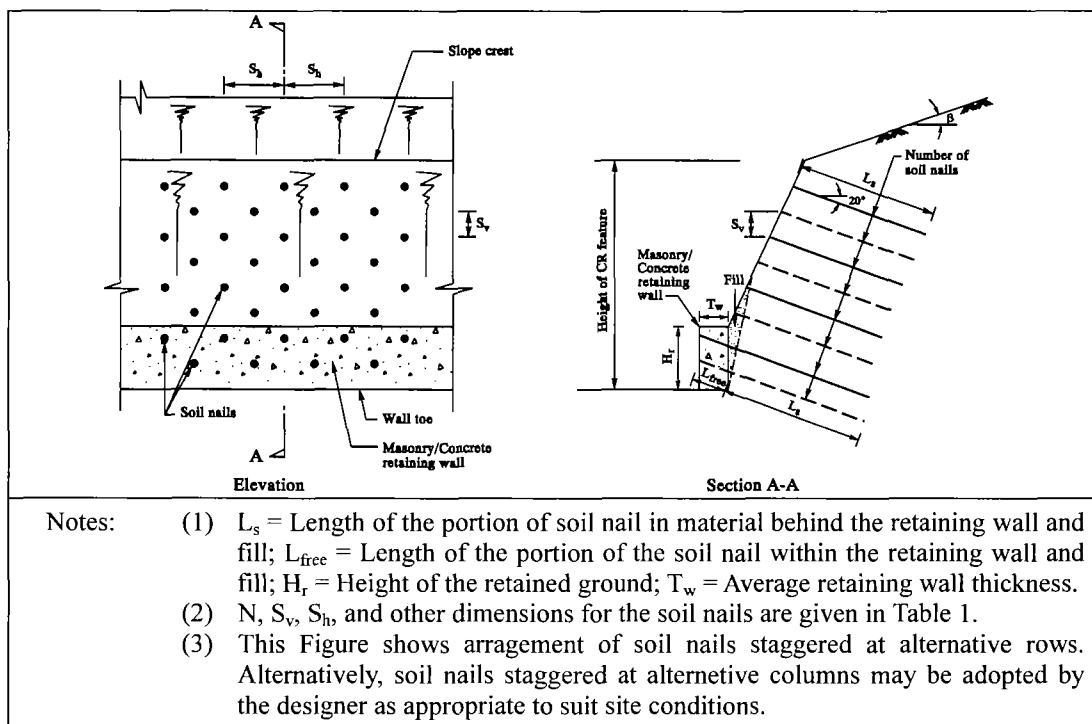


Figure 11 - Soil Nailing to CR Feature

DESIGN AND CONSTRUCTION CONSIDERATIONS

Various design and construction aspects need to be considered when applying the prescriptive soil nail designs to CR and R features. They are discussed below.

- Aesthetics** – GEO (2000) gives guidelines on landscape treatment to man-made slopes and retaining walls. If no skin wall is constructed over an existing wall face, care should be exercised to avoid staining of the surface of the existing wall due to grouting of soil nails.
- Drainage** – Existing weepholes should be extended through any new skin wall. Prescriptive raking drains should be provided to control the rise of groundwater level as needed. Raking drains should be installed after all the soil nails at levels higher than the raking drains have been grouted.

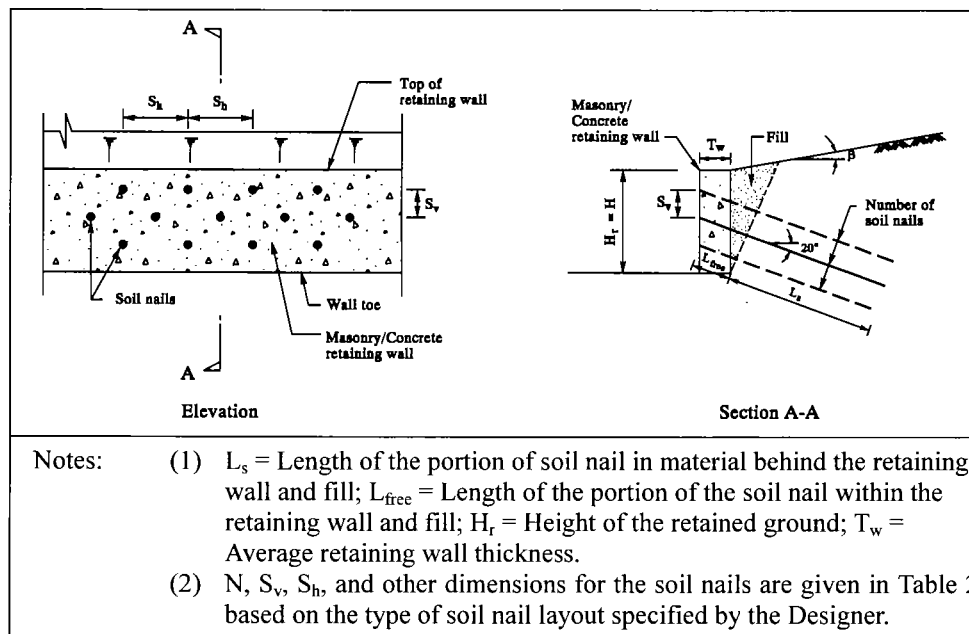


Figure 12 - Soil Nailing to R Feature

- Precautions for drilling through reinforced concrete wall – Before drilling works in a reinforced concrete wall is carried out, precautions should be implemented to avoid damaging steel bars in the reinforced concrete wall, such as using metal detector to determine locations of steel bars.
- Settlement – For soil nails to be installed in a retaining wall with fill material behind, the likelihood of ground settlement at the wall crest and potential adverse effects on exiting utilities and other facilities should be assessed. Staggered drilling of soil nails and provision of casings in drillholes are examples of means of minimizing settlement.
- Grout loss and hole collapse – Considerations should be given to providing temporary or permanent casing in the part of drillholes within the fill material for preventing collapse of the drillholes. Records of grout volume intake should be kept.
- Long and closely-spaced soil nails – Long soil nails are difficult to construct. For the soil nail arrangements listed in Tables 1 and 2, the maximum length of the soil nails is 20m. Besides, the minimum spacing of soil nails is limited to 1.5m.

CONCLUSIONS

Prescriptive soil nail designs have been developed for improving the stability of concrete/masonry retaining walls and cut slopes with a toe wall. Guidelines on the scope of application and design procedures of the prescriptive soil nails are presented.

REFERENCES

- GCO (1984). *Geotechnical Manual for Slopes. (Second edition)*. Geotechnical Control Office, Hong Kong, 301 p.
- GEO (2000). *GEO Publication No. 1/2000 - Technical Guidelines on Landscape Treatment and Bio-engineering for Man-made Slopes and Retaining Walls*. Geotechnical Engineering Office, Hong Kong, 146 p.
- Lui, B.L.S. & Shiu, Y.K. (2004). *Prescriptive Soil Nail Design for Concrete and Masonry Retaining Walls*. Geotechnical Engineering Office, Hong Kong, 76 p. (GEO Special Project Report No. SPR 2/2004)
- MMBP & VLA (1999). *Development of Improved Guidelines for Prescriptive Soil Nail Design*. Final Report submitted to the Geotechnical Engineering Office under Consultancy Agreement no. GEO 13/97.
- Wong, H.N. and Ho, K.K.S., Wong (1995). *General Report on Landslides on 5 November 1993 at Man-made Features in Lantau*. Geotechnical Engineering Office, Hong Kong, 78 p. plus 1 drawing. (GEO Report No. 44)
- Wong, H.N. and Pang, L.S. (1996). *Application of Prescriptive Measures to Soil Cut Slopes*. Geotechnical Engineering Office, Hong Kong, 52p. (GEO Report No. 56, First Edition)
- Wong, H.N., Pang, L.S., Wong, A.C.W., Pun, W.K. & Yu, Y.F. (1999). *Application of Prescriptive Measures to Slopes and Retaining Walls*. Geotechnical Engineering Office, Hong Kong, 73 p. (GEO Report No. 56, Second Edition)

ACKNOWLEDGEMENTS

This paper is published with the permission of the Head of the Geotechnical Engineering Office and the Director of Civil Engineering, Government of the Hong Kong Special Administrative Region.

FIRST USE OF A LARGE DIAMETER SLURRY TUNNEL BORING MACHINE IN HONG KONG

David Salisbury¹ and David Hake²

Abstract: The Drainage Services Department (DSD) of the Hong Kong SAR Government commissioned the design and construction of the West Kowloon Drainage Improvement, Stage 2, Phase 2, Kai Tak Transfer Scheme (KTTS) project in 1999 to relieve potential flooding in Mong Kok, caused by the insufficient capacity of the existing storm water drainage system. The KTTS involves the construction of a 1.5km, 4.4m diameter drainage tunnel and a 450m long box culvert, running from Waterloo Road in Kowloon Tong to the existing Kai Tak nullah in San Po Kong.

This paper describes the design and construction of the bored tunnel, through both hard rock and soft ground, using a slurry Tunnel Boring Machine (TBM); six associated access shafts; an interception structure at Waterloo Road and the box culvert, connecting the bored tunnel to the outfall at Kai Tak nullah. Details will be provided of the contractor's alternative designs adopted during the contract, including, the use of fibre reinforced shotcrete as one-pass temporary and permanent support for three of the shafts and modifications to the Engineer's design for the box culvert structure.

INTRODUCTION

The increased urban development of the Kowloon peninsular, especially the extended reclamation along the western side of Kowloon, has increased the risk of flooding in the low lying urban area to an unacceptable degree. As part of their overall strategy to improve the capacity of the existing storm water system in West Kowloon, the Drainage Services Department (DSD) of the Hong Kong SAR Government, commissioned the design and construction of the West Kowloon Drainage Improvement, Stage 2, Phase 2, Kai Tak Transfer Scheme (KTTS) project in the year 1999. The KTTS forms a critical part of these improvements. It intercepts large stormwater run-off from the Kowloon Tong area and the hills above, which passes into an existing nullah beneath Waterloo Road, and diverts the majority of the peak flows to the east, beneath Kowloon City to discharge into the larger Kai Tak nullah in San Po Kong.

Following a fifteen month pre-qualification and tendering exercise, the HK\$382million design and build (D&B) contract DC/99/05 was awarded to the China Harbour - Transfield Joint Venture (CHTJV) in March 2001.

The joint venture was formed between China Harbour Engineering Company (Group), a State owned Company and Transfield Pty Ltd, a privately owned Australian company.

The detailed design of the scheme was undertaken partly under the D&B contract and partly as an Engineers design. Ove Arup & Partners Hong Kong Limited (Arup), who had assisted CHTJV in the development of the tender submission, were appointed by CHTJV for the detailed design of the permanent works for the shafts and tunnels, and the temporary works for the entire project. Consulting Engineers Black and Veatch Hong Kong Limited (BV) were appointed as the Supervising Officer (SO). BV were also responsible for the detailed design of the cut and cover box culvert section and the hydraulic design. Charles

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² Project Manager, China Harbour Transfield Joint Venture.

Haswell and Partners Hong Kong acted as the Design Checker.

Following successful negotiation of a Contractor's proposed variation, the design responsibility for the majority of the box culvert permanent works was passed to CHTJV. Arup were commissioned to provide the revised detailed design including modifying a large section of the twin cell box culvert to a single cell and pipe arrangement, and optimising the reinforcement of the Engineer's design where possible.

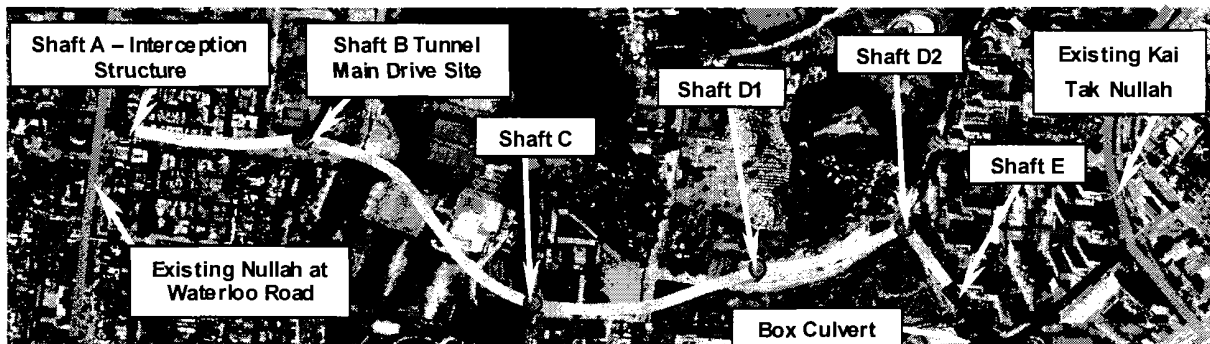


Figure 1. Project Alignment

SCOPE OF WORKS

The KTTS Project is made up of four main components, see Figure 1;

- An interception structure at the junction of Waterloo Road and Hereford Road in Kowloon Tong. This is a cast in-situ reinforced concrete structure connecting the existing decked nullah in Waterloo Road, via a cascade into Shaft A where it connects to the main tunnel.
- A 1.4Km long, 4.4m diameter bored tunnel constructed by Tunnel Boring Machine (TBM) using a precast, reinforced concrete, segmental lining with a cast in-situ invert walkway and dry flow channel.
- Six associated shafts; A, B, C, D1, D2 and E, for access to the tunnel and to facilitate construction of the bored tunnel. The shafts differ in their general arrangement and construction method. Shafts A and E are reinforced concrete box structures, connecting to the interception structure and the box culvert respectively. Shaft A also contains a 4.0m square isolation penstock at the head of the system. Shaft B was used as the main tunnel drive shaft. The TBM was launched from Shaft B, first downstream to Shafts C, D1, D2 and E. It was then dismantled and returned to Shaft B for a final drive upstream to Shaft A. Shafts B, C, D1 and D2 are all circular, constructed within circular (Oval in the case of Shaft B) cofferdams. The base and benching at tunnel level is cast in-situ reinforced concrete. The main shaft lining for these four shafts is sprayed concrete with a float finish, this method is described further below.
- The box culvert is a 450m long cast in-situ reinforced concrete structure. The first 300m being a single cell 4m x 2.8m with a 2.6m diameter pipe alongside, for the majority of its length, carrying existing flows from the Kowloon City area. The box culvert then becomes a dual then triple cell structure as it approaches the outfall at Kai Tak Nullah.

As well as these main items there are associated utility diversions, ventilation columns, a leaky feeder communication system, hydraulic performance monitoring equipment, revised traffic signal ducts, relining of the Waterloo Road nullah upstream and downstream of the interception structure, reinstatement roadworks and landscaping.

ALIGNMENT

The vertical alignment is a constant grade of approximately 0.1% and is fixed by the invert level of the existing drainage structure in Waterloo Road, the hydraulic requirements of the cascade in the interception structure and the outfall level at the Kai Tak Nullah in Lok Sin Road.

The horizontal alignment had to meet the project constraints requiring the tunnel to remain under Government land such as roads and parks. The Employer's scheme design, produced for the tender, had a proposed alignment with a 6m wayleave either side of its centreline. This allowed the Contractor some minor adjustment of the Employer's alignment to suit the construction parameters of the TBM. The horizontal alignment passes from shaft A at Waterloo Road, beneath Hereford Road, to the Junction of La Salle Road where the main worksite was provided adjacent to Shaft B. From here it passes beneath Kowloon Tsai Park to Shaft C at the junction of Inverness Road and Dumbarton Road. The Alignment then follows Dumbarton Road, crossing Junction Road and into Carpenter Road Park where it swings north, through Shaft D1 to Shaft D2 at the junction of Tung Tau Tsing and Tung Tsing Roads. At Shaft D2, the alignment turns 90° to follow Tung Tsing Road to Shaft E where the box culvert starts. This continues down to the end of Tung Tsing Road where it again turns 90° to follow Lok Sin Road to the outfall at Kai Tak Nullah.

The original route proposed at scheme design continued straight across Carpenter Road Park and under the Kowloon Walled City Park. This site is a historical monument owned by Leisure and Cultural Services Department who objected to the proposed excavation beneath the site. This led to a revised alignment skirting the north-east sides of the park and the inclusion of an additional shaft, hence the shaft names D1 and D2.

Curves of 245m minimum radius were used to achieve this horizontal alignment with two points of inflection and a number of straight sections between.

As site B was the only location with sufficient area for the tunnelling operation, this required launching the TBM from Shaft B twice, tunnelling both downgrade and upgrade. The TBM was also turned through 90° at Shaft D2, using a reduced back-up and umbilical connections to excavate the short drive from Shafts D2 to E, this is described in more detail below.

GEOLOGY

The geology along the alignment is highly variable and includes completely decomposed granite (CDG), highly, moderately and slightly decomposed granite (HDG, MDG & SDG), alluvium and CDG with corestones. The crown of the tunnel also comes within 1m of fill material at the crossing of Junction Road; see Figure 2.

Each tunnel section between shafts generally has defined geological conditions as described below in the order of excavation:

B to C – CDG initially, giving way to HDG/MDG entering Kowloon Tsai Park before entering a zone of approximately 270m of SDG to fresh granite, the section also includes a highly weathered fault section identified as a photolineament on geological maps and a basalt dyke.

C to D1 – CDG with interface to alluvium categorised by silty to clayey sands and CDG, CDG & MDG, SDG interface.

D1 to D2 – CDG with corestones.

D2 to E – CDG and alluvium categorised by silty/clayey sands and sandy clays. Very shallow cover of less than 0.75 tunnel diameter approaching Shaft E.

B to A – CDG with some corestones and slightly decomposed granite returning to CDG then alluvium with peat deposits approaching Shaft A.

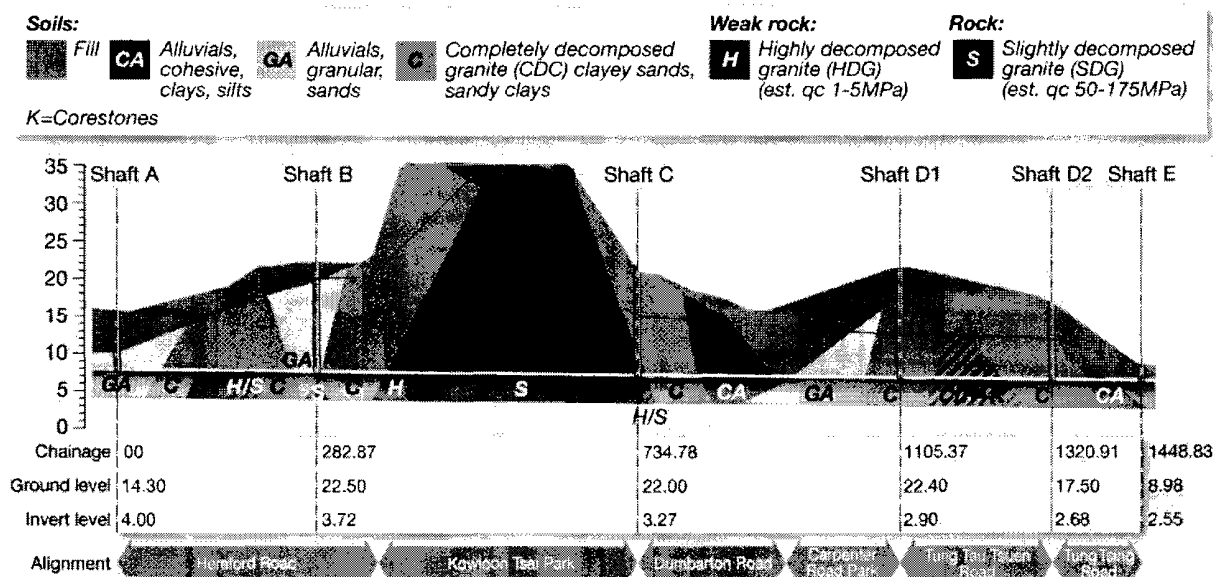


Figure 2. Simplified Geological Section

All sections of tunnel are beneath the water table with a maximum hydrostatic head in the soft ground sections of 1.4bar at crown level. Details of the compressed air working required for TBM maintenance at these pressures are given below.

DETAILED DESIGN

Arup were responsible for the detailed design of all temporary and permanent works for the KTTS interception structure; shafts; and tunnel. They were also required to provide an assessment of the effect of the works on existing buildings and structures along the route.

Design Approval Process

The project was exempt from Buildings Department approval as the Client, DSD, are a government department themselves. The design submission process included consultation with all interested third parties including the SO; DSD divisions; utility companies; FSD; Police; WSD; Highways; other government departments; district boards.

The design followed an Approval in Principle (AIP), followed by a Detailed Design Approval (DDA), process. To streamline the design process the scope of works were separated into a number of temporary and permanent works design packages. More than 90 design packages were produced in total, each one distributed to relevant interested parties for comment, including the SO and Design Checker. The consolidated comments were addressed and redistributed for further comment or approval before producing a revised document, which was submitted to the Design Checker and then the SO for certification. The design approval flowchart is given in Figure 3.

Despite the large number of submissions involved, this process proved to be relatively streamlined with the main co-ordination for the documents being handled by the CHTJV Engineering and Approvals Manager.

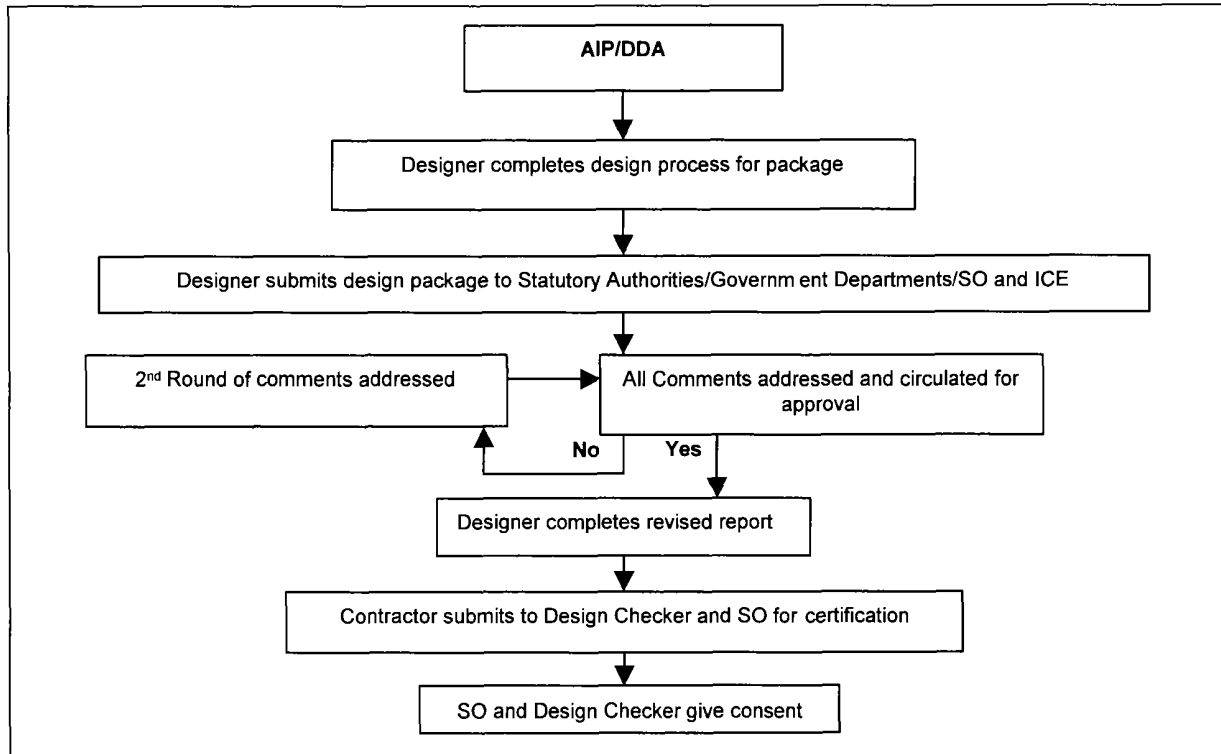


Figure 3. Simplified Design Approvals Flowchart

Design Changes

The original design scope, as outlined in the Employer's requirements (ER), was modified during the construction of the works. These modifications were, usually, Contractors alternatives and were adopted where either; a saving to the cost of the project could be made or, where an enhancement to the quality of the works could be demonstrated. The major permanent works design changes were as follows:

1. Interception Structure

- The interception structure underwent several minor modifications to the original internal dimensions given in the Employer's drawings. The SO approved all these modifications, as they did not affect the hydraulic performance of the interception structure. The original hydraulic design had been tested using a 1:25 undistorted Froude scale hydraulic model at the University of Hong Kong. The model was retained to allow for any major modifications to the design to be tested, which may have been required.

2. Shafts

- Shaft A was modified to a reinforced concrete box structure of a similar internal area to the 7.0m diameter minimum required in the ER.
- Shaft B temporary works were modified to an oval shape to allow the TBM back-up decks to be installed. Despite difficulties with the design and construction, this arrangement proved valuable when the decision was taken to launch the TBM from within the shaft, rather than construction short stub tunnels as launch chambers on either side of the shaft. The additional size of the temporary works allowed the permanent works vortex drop shaft, for a 1.35m incoming storm drain, to be relocated on the outside of the main shaft

structure, thus allowing the shaft permanent works to be reduced to the minimum 7.0m diameter requirement.

- Shafts C and D1 were modified to incorporate an access from the lower landing through the top of the segmental tunnel lining. This avoided the need for the majority of the reinforced concrete benching at the base of the shaft. The permanent shaft lining of these two, and shaft D2, were constructed using fibre reinforced sprayed concrete.
- The position and size of Shaft D2 was modified to accommodate the construction method and the turning of the TBM for the revised alignment between shafts D2 and E.
- Shaft E was repositioned approximately 30m North to suit the revised alignment between Shafts D2 and E, and to achieve better ground cover for the TBM. The shaft was modified to a reinforced concrete box to link the bored tunnel to the extended box culvert.

3. Box Culvert

- Arup redesigned the box culvert, originally undertaken by the Engineer BBV. Two additional box culvert sections were added, E1 and E2 to extend the box culvert to the revised shaft E location. Bays 1 to 13 were redesigned to reduce the reinforcement without modification to the internal dimensions. Bays 14 to 34 were modified from a dual cell box culvert to provide a single box culvert for the KTTS flow and a parallel 2.55m ID pipe into which the existing flows from Lok Sin Road were diverted. The outfall structure between Bays 35 and 38 were constructed in accordance with the ER.

Tunnel Lining Design

The lining is a single-pass “universal taper” lining, 4400mm internal diameter, 250mm thick, nominally 1200mm in length with a 28mm taper. The precast ring is made up of six pieces, and has a two part Ethylene Propylene Diene Monomer (EPDM) and hydrophilic strip insert gasket to achieve the water tightness requirements. The circumferential joint uses galvanised dowels, whereas the radial joint has spear bolts (removed at the rear of the back-up) and each segment has a standard central lifting socket.

The preliminary lining design was considered under elastic loading with factored loads. It was assessed using continuum analysis in an isotropic medium to Curtis (1976) / Muir-Wood (1975) methods using Duddeck and Erdmann (1982) formulae.

Detailed analysis of critical sections of the tunnel was carried out using finite difference modelling with the use of the FLAC computer programme.

The segments were made from grade 50 concrete with a w/c ratio of 0.4, 25% PFA and 6% silica fume. A test to achieve 1×10^{-12} m/s maximum permeability was added to the standard testing requirements. The segments were designed with 40mm minimum cover to reinforcement. All these measures were adopted to enhance the durability of the lining.

The precast concrete lining was manufactured in China using French-made moulds and shipped to Hong Kong by barge.

The lining has all lifting sockets and bolt pockets backfilled with a cement based mortar to ensure the hydraulic smoothness for the 43m³/s maximum design flow in operation.

The tunnel invert has a secondary insitu concrete roadway with an asymmetric low flow channel, using standard steel formwork panels and concrete was pumped up to 350m from shafts D2, C and B.

Durability

The shaft and tunnel works are designed for a service life of 120 years. The shafts and tunnels have been designed for “very severe” conditions as defined in the Highways and Bridges Design Standards Manual, Table 20.

Suitable protective measures and precautions against corrosion were developed during the detailed design to meet the design life requirements. These included;

- Permeability test on precast segments,
- Silica Fume added to concrete mix,
- Unreinforced concrete shaft linings,
- 0.15mm crack width design for reinforced concrete,
- Float finish to shotcrete lined shafts.

Other items such as ladders, steel landings and ventilation columns have a 25 year design life.

TEMPORARY WORKS DESIGN

Interception Structure

Waterloo Road Interception Structure temporary and permanent works were designed in four stages to allow the temporary traffic diversions necessary to maintain three southbound and two northbound lanes in Waterloo Road at all times.

The four stages included piling to support two southbound carriageways on a temporary steel deck, temporary walls using sheet piles; pipe piles; H-piles and lagging with Tube-A-Manchette (TAM) grouting to reduce the permeability of the walls and provide water cut-off at the toe of the sheet piles.

The prestressed beams over the existing nullah were cut and removed, one side of the nullah wall was removed and a reduced flow temporary wall was installed in the nullah. This was allowed to remain in place throughout the 2003 wet season as long as the equivalent flow of a 2.8m diameter pipe was maintained at all times.

Steel struts and walings were used in a complex arrangement to support the temporary walls. This was made more difficult because of the staged construction of the works.

Shafts A and E

Both shafts were designed as square sheet piled shafts with toe grouting to provide hydraulic cut off. Both shafts had to provide a clear vertical opening between the base slab and the lowest waling for the TBM to break into. A temporary bracing level at TBM level, which could be removed once the temporary works base slab had reached its design strength, achieved this. The shafts also had to provide enough plan space between the walings, cross struts and corner braces for TBM dismantling and removal.

Shaft B

Originally designed as a circular pipe pile shaft, Shaft B evolved into a 10.5m x 9.5m elliptical sheet pile shaft with two cross struts obstructing the main body of the shaft during excavation. The change in shape was due to the shaft size requirements for the TBM back-up decks and the constraints of major utilities on either side of the shaft. Early difficulties with settlement of the surrounding ground when pipe piling was commenced drove the change to vibro-hammer piling sheet piles to 20m depth then toe grouting. The cross struts were removed once the shaft base had been concreted leaving only ring beam walers around the shaft perimeter to allow full access for the TBM installation. The shaft layout included the breakout seal ring attached to a shotcrete launch stub, thrust frame, slurry pump and a window for lowering segments.

Shaft C

Due to the presence of rock and corestones at Shaft C, the 8.5m diameter, temporary works were designed using 406mm diameter steel pipe piles. These were installed 25m deep using a down the hole hammer with compressed air flushing. TAM grouting was used to seal the gaps between the piles against water ingress. The pipe piles were backfilled with concrete with the exception of the eyes for the TBM breakthrough, which were backfilled with stabilised sand.

Shafts D1 and D2

These were both designed as circular sheet pile shafts with circular walings for temporary support as the excavation progressed. A reinforced concrete benching and unreinforced concrete cast in-situ permanent lining would follow completion of the TBM tunnel. The alternative design incorporated a sprayed concrete lining forming both the temporary and permanent works installed in a top-down method. The construction joints were formed as staggered joints with grout tubes and hydrophilic strips installed to allow grouting of the joints should any leaks appear in the lining. The construction method is described in more detail below.

Box Culvert

The cut and cover works along Lok Sin Road and Tung Tsing Road followed the common approach of a strutted sheet pile cofferdam with H-pipe and lagging, and TAM grouting to cross major utilities.

Design Software

The majority of the temporary works were modelled using the following design software packages.

Oasys FREW for the sheet pile walls;

Oasys GSA for the strut, waling and reinforced concrete permanent works;

Oasys SEEP for the ground water drawdown analysis;

Oasys TUNSET for tunnel settlement analysis;

Arup In-house spreadsheets for the tunnel lining design (FLAC used to verify critical sections);

FLAC for the ribs and sprayed concrete lining of the TBM launch chambers at shafts B and D2. (D2 was the only one constructed).

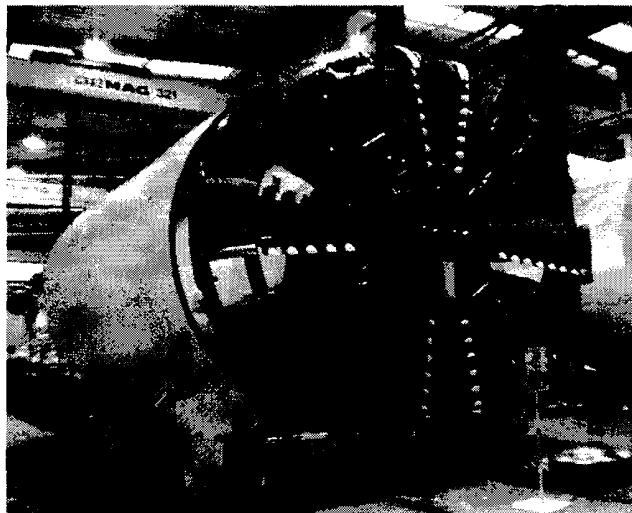


Figure 4. Herrenknecht TBM

TUNNEL CONSTRUCTION

TBM & Equipment

Due to the varied ground conditions and settlement issues, CHTJV selected a refurbished 5.17m diameter Herrenknecht mix-shield TBM, which had been used on Jubilee Line Extension in London and at Ayr in Scotland, on a rental agreement from Herrenknecht. (See Figure 4).

The design of the cutter head included 24 x 14" (355mm) single cutters capable of 20T per cutter, 4No twin cutter sets for rock conditions and 50 soft ground scrapers for soft ground.

Excavated material is mixed with a bentonite slurry which is continually pumped into and out of the cutterhead. The slurry/ground mix is pumped along 250mm diameter pipelines, back to the surface where it is separated through a screening plant, the bentonite being recirculated to the TBM in a continuous process while excavating.

The TBM uses a compressed air "bubble" behind the top of the cutterhead to assist in balancing the external ground pressures and acting as a damper to the continually changing thrust load pushing against the ground. The "bubble" was maintained by alternating operation of two electric compressors located within an acoustic enclosure, provided with high quality filtration and a chiller unit for compressed air interventions into the cutterhead to change tools. A water chiller system with high pressure supply pumps and a cooling tower were required for the TBM cooling water. The segmental lining is erected immediately behind the main TBM shield and this ring then provides the reaction to the 16, 200T propulsion rams.

Tunnel Excavation

The TBM was launched at Shaft B. The shaft layout included the breakout seal ring attached to a shotcrete launch stub, thrust frame, slurry pump and a window for lowering segments.

CHTJV worked closely with Herrenknecht for the development of the short, staged launch and use of umbilical lines for TBM systems (See Figure 5). The short launches required the TBM to commence without the back-up. Therefore to connect the TBM to the backup, "umbilical" lines were required for the slurry system, hydraulics and electrics.

The umbilical launches resulted in a three stage process as follows:

Stage 1 - advance to limit of the slurry pump suction pressure to install the bridge, segment feeder and gantry deck 1, requiring temporary systems for segment handling to the erector.

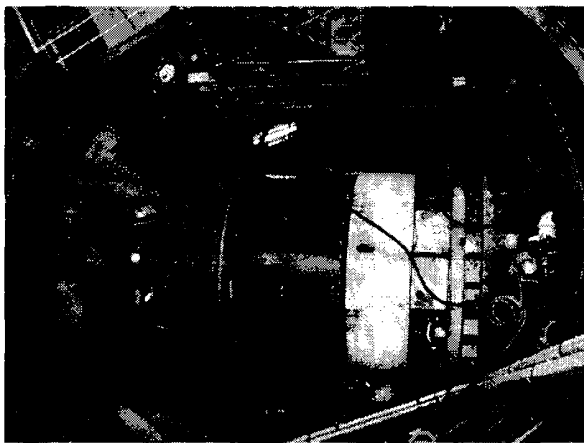


Figure 5. TBM Short Launch in Shaft B

Stage 2 – advance with gantry deck 1 installed to develop tunnel length for installation of the remaining backup, requiring temporary systems for segment transport.

Stage 3 – advance with full back-up installed, including telescopic pipes for slurry line extension.

The electrical and hydraulic umbilicals were carried forward as the TBM advanced using 100UC sections suspended off

support brackets from the lining and girder trolleys with soft slings on either side of the tunnel.

The first 100 rings were excavated in soft ground with negligible wear, however the TBM entered the rock section where the machine performance reduced dramatically in the slightly decomposed to fresh granite.

TBM Performance

CHTJV, with the support of Herrenknecht, focussed on improving the TBM's performance by increasing the cutter head RPM by 25% and replacement of the main drive displacement cylinders to increase the contact force by approximately 45%. Advance rates improved after the TBM modifications were completed.

To negotiate the tight radius curve, the pre-gauge and gauge cutters were offset with cutter seat inserts to provide greater clearance around the curves. The disadvantage of the extra over-cut was grout control or loss to the slurry circuit and potential ring movement/damage due to ring flotation within the liquid grout annulus. An accelerated high cement grout mix was pumped through the lining in the crown and shoulders to stabilise the lining (in the rock section) and followed-up with periodic back-grouting.

Cutter usage on standard 14"(355mm) rings in the rock was high, however this improved after replacing the rings with heavy duty (HD) alternatives. Cutter changes in the rock section were undertaken in free air with consideration to achieving an initial set of the annulus grout before the pressure in the slurry system was reduced.

The high loadings and hard work in the rock section took its toll, resulting in cracking of the cutter head, drive plate and displacement cylinder supports which required additional stiffeners to complete the rock section between Shafts B & C. Additional refurbishment was carried-out at Shaft C.

The TBM excavated successfully in difficult interface conditions adjacent to Shaft C, with 50% rock and corestone and 50% CDG with only 2.5x diameter cover and high water table. Advance rates were reduced to minimise cutter and tool damage and to minimise regenerative noise issues for buildings adjacent to the alignment.

After the low cover intersection crossing at Junction Road, the TBM had a 'safe' run beneath Carpenter Road park in uniform CDG ground conditions where all systems were pushed for production and 13 excavated rings (15.6m) in one shift was achieved in this period.

At Shaft D1, the cutter head was prepared for rock and corestone between D1 and D2 and this was realised within 40m of leaving D1. The granite transitions very quickly over a narrow band from CDG to SDG, which can result in cutter damage through impact loading. The section between D1 and D2 also included passing adjacent to a sensitive existing structure, Mei Tung House. This is a 10 storey housing block, built in the mid '70s on pad footing foundations. A full structural assessment and analysis was required to satisfy Housing Department and the estate management that the settlement effect on this building could be effectively controlled. In the event the settlement in this area was around 20% of the predicted values, causing negligible movement to the building.

After 10.5 months in the ground, the TBM broke through into Shaft D2 in August 2003.

Turning of TBM 90° at Shaft D2 & Re-Launch

The project alignment required a 90° corner at Shaft D2 and a solution was required which did not disrupt traffic and would still be workable within a tightly constrained site (See Figure 6).

To achieve this, the following was required:

- Increase the shaft internal diameter to 9.5m.
- Jet grout a zone and construct a launch chamber using shotcrete and lattice girders in water bearing CDG.



Figure 6. 90° bend at Shaft D2

- Develop grease-injected “hovercraft” plates to be incorporated onto the existing TBM cradle and install a steel base plate for turning the TBM at 185T, excluding the tail shield.

The above allowed the TBM to be turned mostly intact (except the tail shield) which reduced the duration of the turning and re-launch process, without disruption to traffic as no road closures were required for heavy crane lifts and the tail shield lift could be undertaken safely from within the site area.

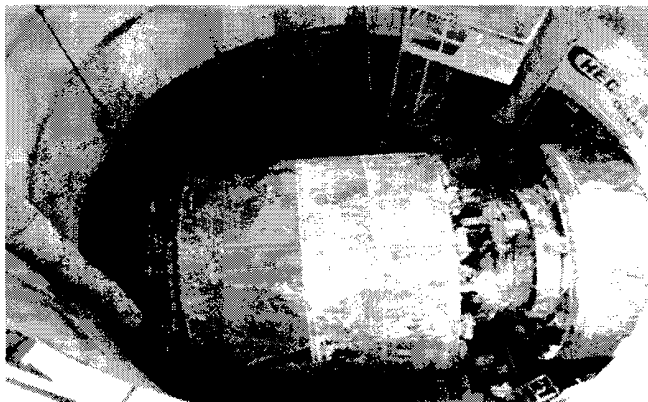


Figure 7. TBM Separation at Shaft D2

The shaft base was prepared by placing 20mm plate within the locus of planned movements of the TBM cradle. The plates were welded together, anchored to the slab, all imperfections removed. The TBM cradle was prepared with the addition of 3 x 50mm plates on the underside support beams. The plates had been prepared with grease injection ports and machined recesses for seals provided by an o-ring and PTFE ring.

The TBM was separated from the backup and pushed onto the cradle using a power pack connected to the TBM’s bottom thrust cylinders (See Figure 7). With the TBM in the correct position, the cradle was released and



Figure 8. TBM Being turned

using spacers, the TBM pushed itself and the cradle forward to the shaft wall. The TBM was pulled diagonally across the shaft using a hydraulic cylinder to allow the erector beam to clear the tail shield. Then the second pull point was used to pull the rear of the TBM onto the correct alignment (See Figure 8). The cradle was then welded to the base plates and the TBM pushed forward into the launch tunnel using the same power pack and bottom thrust cylinders.

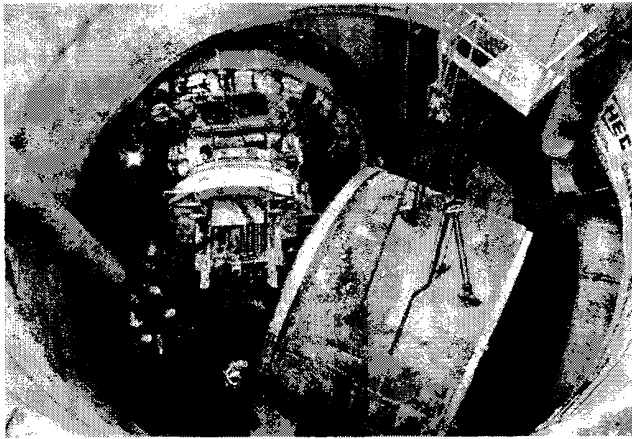


Figure 9. Tail shield being turned

The tail shield was lifted and turned conventionally using a 160T crane from within the working site. A stoppage of about four weeks was required to turn the TBM and recommence excavation (See Figure 9).

After the completion of stage 1 excavation (approx. 30m), gantry 1 was turned in the shaft, requiring the top section to be removed and fixed in the tunnel crown. The lower section was winched into the shaft and turned using a crane and winched into the tunnel. The top was 30% longer than the base and hence the top was turned on its side to fit

around the corner, then rotated back flat, lifted into the crown and reconnected to the lower section.

The 90m tunnel section between Shafts D2 and E was controlled by planned stoppages rather than excavation rate, whereby after re-launch and 1.5 weeks excavation the TBM was stopped for 2.5 weeks to turn gantry 1. Excavation was re-commenced and the drive completed after a further two weeks. Breakthrough at Shaft E was achieved in late October 2003.

Close surface settlement monitoring was carried out over this length of tunnel as the cover reduced from 1.1x diameter down to 0.8x diameter under an operating road.

The TBM and back-up was transported back to Shaft B passed the previously excavated Shafts D2 and D1, requiring site D1's hoarding to be removed as the cutter head and shields were transported. Transport restrictions for the over-size and over-weight components required transportation over four nights.

Drive B to A

The TBM was re-launched at Shaft B in early December 2003, only 6.5 weeks after breakthrough at Shaft E. The final 280m drive to Shaft A was also complicated by varying geology (including a small rock-head), low clearance to major utilities; including, a 1350mm

diameter water main only 3.0m above the crown of the TBM and low cover below an existing road, adjacent to sensitive residential structures.

The final breakthrough at Shaft A came on 26th February 2004. The TBM was dismantled and removed back to the Shaft B site where it was packed for shipping to Guangzhou for storage.

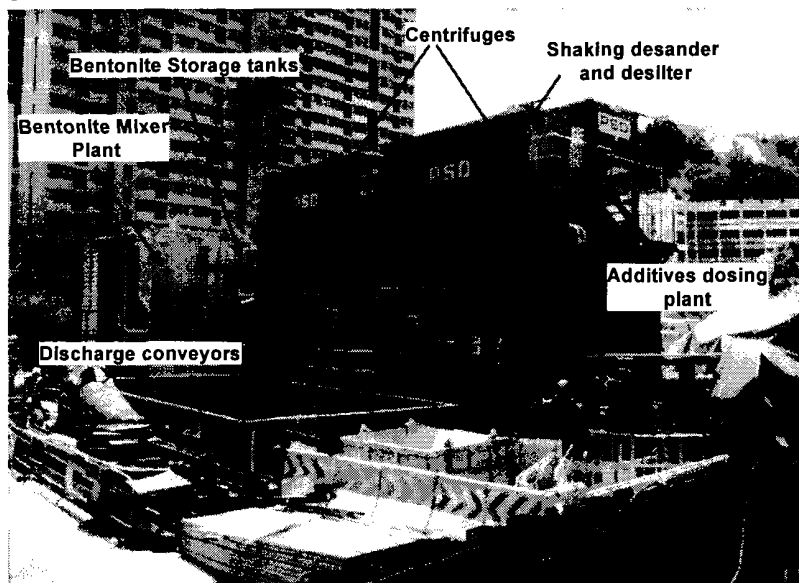


Figure 10. Slurry Treatment Plant

Slurry Treatment Plant & Spoil Handling

CHTJV selected Pigott Shaft Drilling Ltd (PSD) from the UK as the supplier for the slurry treatment plant (STP) under a rental agreement and utilised their in-house expertise for the selection of bentonite and flocculants. The STP consists of a four stage separation plant of primary shaker, desander, desilters and two centrifuges with a total capacity in excess of 750m³ per hour (See Figure 10).

Bentonite was added to the slurry system using either a high shear mixer to produce fresh bentonite slurry or via a venturi jet hopper to add powder directly to the active system.

During tunnelling operations, minor issues such as; foaming due to cement contamination when excavating through ground treated zones and foamed concrete backfill, were addressed quickly through additives and none of these issues limited the TBM's performance.

A further key consideration was the STP performance for separation of the clays and silts within the CDG and maintaining an operable slurry density throughout the shift. To this end the PSD system has proved particularly effective across the varying ground conditions and was very reliable with an amazing 0% down time against the TBM utilisation time.

SETTLEMENT MONITORING

Settlement monitoring was carried out by precise levelling in the vicinity of the TBM using standard ground settlement points along the alignment centreline and specific utility monitoring points located on sensitive services.

The surface monitoring provided feedback into the model for TBM operating pressures and grout pressures to minimise surface movements to generally less than 0.2% face loss.

The TBM completed and passed three major tests for face control;

- Low cover tunnelling beneath Junction Road with less than 1x diameter of cover to utilities and two diameters to a busy traffic intersection;
- Low cover tunnelling between Shafts D2 and E with 0.8x diameter of cover; and
- Low cover tunnelling beneath a 1350mm diameter fresh water main for a sub-parallel crossing with 0.6x diameter clearance over a 30m length of tunnel.

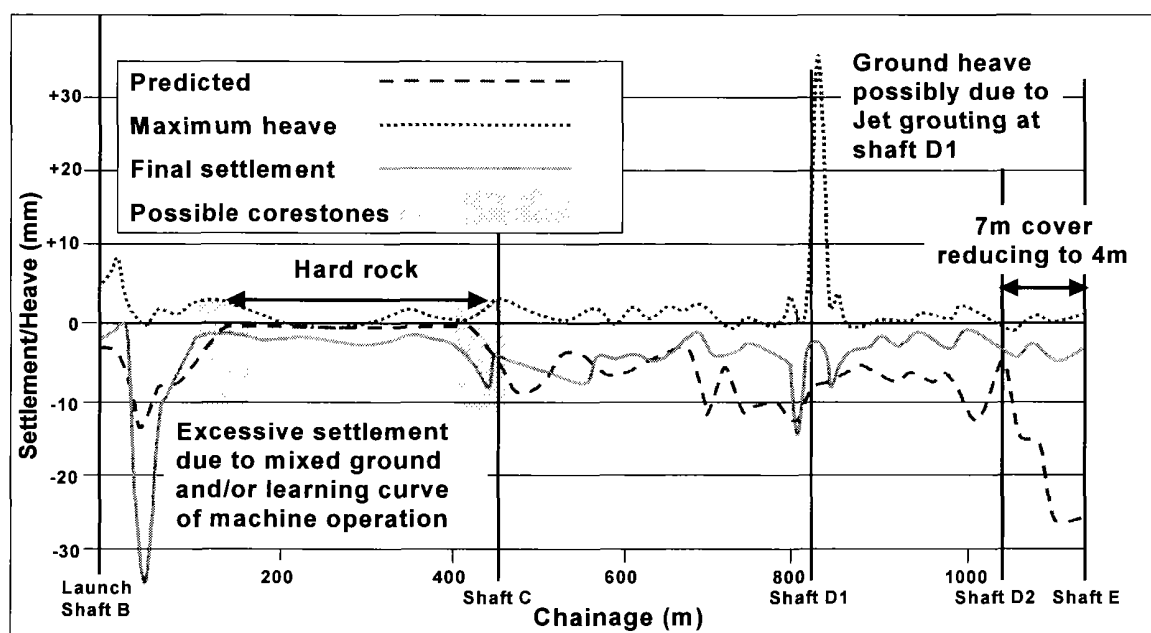


Figure 11. Ground Movement – Predicted and Actual - Tunnel Drive B to E

The mix-shield proved very effective, with vigilant operation and grouting, for settlement control in areas of low cover, utilities and interfaces of rock and soft ground beneath roads and sensitive adjacent structures.

Some early problems with the erection of the segmental lining were quickly overcome, providing a high quality completed tunnel with no visible leaks within alignment tolerances (See Figure 11).

COMPRESSED AIR & SAFETY

Compressed air (CA) interventions were carried-out for cutter head maintenance in soft ground below the water table. The TBM has a standard interconnected two lock arrangement (See Figure 12), certified up to 3bar. The CA work was undertaken in accordance with the HK Labour Departments requirements Chapter 59 Factories and Industrial Undertakings Ordinance (Work in Compressed Air) Regulations, which uses the UK derived Blackpool tables for decompression.

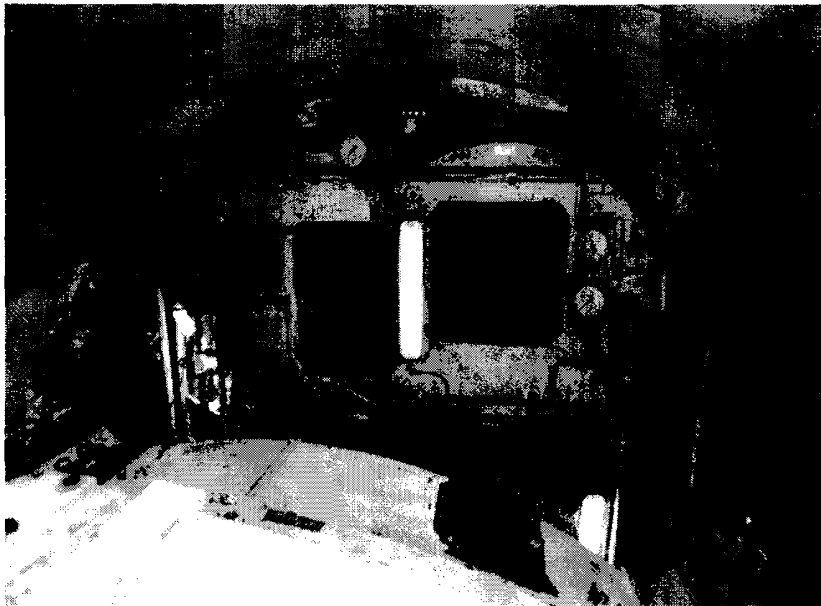


Figure 12. Compressed Air locks in TBM head

The Fire Services Department (FSD) set up an Emergency Pressurisation Team (EPT) capable of entry into the working chamber. This required ongoing pressurisation training of FSD personnel in the surface recompression chamber on a monthly basis for a total of more than 60 FSD personnel.

There were less than 180 man interventions and exposure time on average was less than 2.5hours. Fortunately the tunnel is relatively shallow and the water table along much of the alignment was not high, hence 95% of the compressed air work was undertaken at less than 1bar above atmospheric pressure. Despite this, full medical lock facilities were situated at Site B until the TBM reached Shaft D2.

The TBM used FSD approved fire resistant hydraulic oils, fire resistant vent duct and was fitted with fire alarms, automatic fire detection systems and continuous gas monitoring.

SHAFT CONSTRUCTION

The six shafts constructed along the tunnel alignment provide for DSD and emergency access to the permanent works. The methods of construction for each shaft are varied to suit the ground conditions and constraints at each shaft location.

Temporary Works Construction

The temporary works used a vibro-hammer to install sheet piles 20m deep at shaft B. The sheet piles at the shaft eyes were removed, and the exposed ground shotcrete, after installation of two rows of contiguous continuous flight auger piles for ground stabilisation.

A drop hammer was used to install sheet piles 21m deep at shaft D1 and a Super Crush Silent Piler (auger & hydraulic insertion) to install sheet piles up to 24m long in one pass at shafts D2, E and A.

Due to the presence of rock and corestone at Shaft C, 406mm diameter steel pipe piles were installed 25m deep using a down the hole hammer with compressed air flushing.

One-pass Sprayed Concrete Lining

Following the sheet or pipe pile wall installation and TAM grouting, the circular shafts were completed using a temporary and permanent shotcrete lining placed in two layers, a primary Steel Fibre Reinforced Shotcrete (SFERS) layer and a secondary plain layer (See Figure 13). The joints incorporated a hydrophilic water stops and injection grout tube to ensure water tightness, no waterproofing membrane was adopted. A staggered construction joint was adopted between the two layers using formed shutters to provide a high quality



Figure 13. First stage of One-pass Sprayed Concrete Shaft Lining

construction joint. The face of the second, finishing layer has a F3 float finish as required by the ER. The final product appears to be cast in-situ concrete.

The use of a one pass temporary and permanent sprayed lining is believed to be a first for Hong Kong. As it is installed with the shaft excavation it has also been seen to reduce ground movements whilst providing programme and cost savings. Care has to be taken fixing the construction joint formwork to provide a clean joint. A high early strength shotcrete mix was used to minimise the downtime of excavation waiting for early strength to develop. Test cubes

usually showed that the temporary works strength requirement was achieved in 12-16 hours. The works were being undertaken in daytime only; this meant that, with 12-14 hours overnight and an early morning compressive strength test, excavation could usually recommence the following morning after spraying was complete.

TBM Entry and Exit to Shafts

The mid-drive shafts C and D1 were backfilled with foamed concrete to above tunnel crown level to suit the shaft permanent works intermediate slab level and for the TBM to excavate through. After the TBM was safely beyond the shafts, the foamed concrete was excavated down to the segmental lining and prepared with epoxy fixed dowels for pouring in-

situ walls and intermediate slab. After curing, the segmental lining was removed using wire rope saws, completing the tunnel opening to the shaft.

Shaft D2 was enlarged to 9.5m internal diameter for turning the TBM and also to enable construction of the TBM launch chamber. The 5.5m diameter shotcrete lined launch chamber was nine metres in length and excavated by hand using NATM methods (lattice girders and shotcrete lining) through a jet grouted CDG block.

BOX CULVERT CONSTRUCTION

Extensive temporary traffic diversions were required to maintain traffic flows and achieve the construction programme. The programme was constrained in Tung Tsing Road due to the recent HyD project to upgrade and replace the road. Re-excavation under the KTTS project could not commence until one year after completion of the previous project. The Super Crush Silent Piler and vibro piling were used for the sheet pile installation. The permanent works were, typically, constructed in three stages; cast the base slab, splays and wall kickers; once this had reached the required strength remove the lower temporary prop; construct the walls and roof.

Once complete the utilities, pavement and road surface were reinstated.

ENVIRONMENTAL & COMMUNITY

The main tunnelling site was located in an exclusive residential area in Kowloon Tong and was subject to strict environmental considerations regarding truck movements and noise, restricting TBM excavation to dayshift only.

The nearest sensitive receiver was only 20metres from the main working shaft and a residential university building overlooked the site compound, approximately 40m from the STP. Therefore extensive noise attenuation works were required to minimise the risk of complaints and also to extend available working periods, including enclosing the STP within an acoustic building, enclosing the compressors within a lined building, placing barriers around the water chiller unit, enclosures for the gantry crane motors, barriers for the ventilation intake and the fans were relocated into the tunnel.

Despite these difficult conditions, the works were completed with minimum disturbance and CHTJV's efforts were recognised through the Environment, Transport and Works Bureau's Considerate Contractors Site Award Scheme and the Site Cleanliness & Tidiness Grand Award Scheme.

CONCLUSIONS

The KTTS project provided the first use of a large diameter mix-shield TBM in Hong Kong. It provided excellent settlement control in difficult mixed ground conditions. Typical settlement values equated to a face volume loss of 0.2% were achieved, with more precise control provided where required.

The construction of the interception structure in Waterloo Road was achieved without any incident or disruption to the traffic flow on this major trunk road. The complex construction

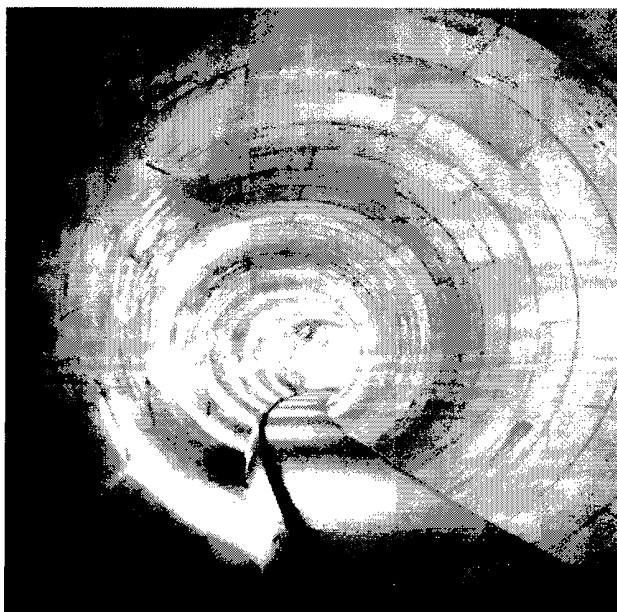


Figure 14. Completed Tunnel

sequence of the temporary and permanent works required detailed planning to link the design to the construction method.

The innovative use of sprayed concrete for the shaft lining has produced a high quality product, which was quicker and cheaper to construct. Sprayed concrete was also used successfully for the relining of the existing Waterloo Road nullah and some of the formed permanent benching works in the shaft inverts.

The close proximity of a large number of residential properties, both private low rise and public high rise, was seen as a major constraint on the project. One of the major successes has been that the project was able to achieve acceptable productivity in most elements of the project while maintaining good discipline in community relations, safety and environmental controls.

REFERENCES

Muir-Wood, A.M. (1975) The circular tunnel in elastic ground. *Geotechnique* 25(1), 115-127.

Curtis, D.J. (1976) Discussion on 'The circular tunnel in elastic ground'. *Geotechnique* 26, 231.

Duddeck, H. & Erdmann, J. (1982) Structural design models for tunnels. *Tunnelling* '82, 83-91.

ACKNOWLEDGEMENTS

The permission of the Drainage Services Department of the Hong Kong SAR to publish this paper is gratefully acknowledged.

Automatic Drilling Process Monitoring for Rationalizing Soil Nail Design and Construction

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Abstract: This paper explores a possible methodology for rationalizing the design and construction of soil nail. The methodology includes an innovative Drilling Process Monitor (DPM) and an associated data analysis method. The DPM can automatically, objectively and continuously measure and record the drilling progress and operation in real time while a hole is being drilled in the ground. The DPM is portable and can easily be mounted to existing drilling machines. The data analysis method is simple and can be carried out with a personal computer. Case studies are presented to show the DPM method and how it can be used in soil nailing. The examples demonstrate that the DPM method could give an indication of the soil and rock (including boulders) profile along a soil nail hole. Such factual results can provide further information for the design of soil nails, to verify a soil nail design and possibly to make design variation for the purpose of safety and cost-effective. Discussions are further made on the possibility of using the DPM method to rationalize soil nail design and construction.

INTRODUCTION

Since 1998, the Jockey Club Research and Information Centre for Landslip Prevention and Land Development of the Department of Civil Engineering, The University of Hong Kong (HKU), has launched and conducted a research and development project in automatic monitoring of drilling process for geotechnical engineering (Lee et al. 1999). This project has resulted in the invention of an automatic digital Drilling Process Monitor (DPM) for ground characterization and construction management in association with drilling (Yue et al. 2001; 2002; 2003a; 2004; Sugawara et al. 2003a; 2003b). The DPM can automatically, objectively and continuously measure and record the drilling progress and operation in real time while a hole is being drilled in the ground. A data analysis method has been developed to analyze the original DPM data for geotechnical use. This paper is intended to introduce a methodology for rationalizing the current soil nail design and construction methods. This methodology consists of the DPM and the associated data analysis method. The feasibility of the proposed methodology is demonstrated by case studies of actual examples associated with automatic monitoring of drilling holes for installation of soil nails in slope stabilization works in Hong Kong.

CURRENT SOIL NAILING PRACTICE

Soil nailing is an in-situ technique of reinforcing soil slopes by horizontally and closely installing unstressed slender inclusions. The unstressed slender inclusions (or soil nails) are usually made of steel bars covered with grout. They have sufficient tensile strength to resist forces induced by possible slope lateral displacements. As a result, a reinforced soil slope can be developed and has an increased overall shear strength to restrain slope lateral movement. Soil nail technique has been developed basically as an extension of the New Austrian tunneling method which combines reinforced shotcrete and rock bolts to

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provide a flexible support for the construction of underground excavations. Records of soil nail applications to engineering projects around the world may be found in publications such as Campanella and Robertson (1985), Gassler (1990) and Juran and Elias (1991).

In Hong Kong, use of the soil nail technique started in 1985 when soil nails were adopted as a prescriptive measure to treat deeply weathered zones in sound rock (Chung, 1992; Powell and Watkins, 1990; Watkins and Powell, 1990). Since then, soil nailing has been widely accepted as an ideal remedial technique to stabilize marginally stable soil slopes and retaining walls in Hong Kong. In recent years, soil nails have been extensively adopted to upgrade existing substandard cut slopes and old masonry retaining walls and new steep cut slopes. In particular, it is roughly estimated that hundreds and thousands soil nails had been installed in more than 500 government slopes and retaining walls during the 5-year Accelerated Landslip Preventive Measures (LPM) Project between 1995 and 2000.

Soil Nail Design

At present, two soil nail design methods are used in Hong Kong. One is the conventional analytically based design approach (Powell and Watkins 1990; Watkins and Powell 1990; Wan and Yue 2003; 2004 and Yue et al. 2003b). The other is the prescriptive design approach (Wong and Pang, 1999). The prescriptive soil nail design method was developed using the results of analytical soil nail design methods for upgrading more than one hundred existing cut soil slopes under the LPM project. The prescriptive design method is then applicable only to existing soil cut slopes with simple geological condition and effective slope height less than 20 m. The analytically based design methods generally do not have such limitations. The procedures of the analytically based soil nail design methods usually cover the following routine steps: desk study, site inspection, ground investigation, laboratory testing, stability assessment of existing slopes, design option assessment, soil nail design and checking, and design verification during construction.

One of the most important aspects in soil nail design is the construction of a geological model that truly represents the ground condition of a particular slope or retaining wall. Factual data that are obtained from the desk study, site inspection, ground investigation and laboratory testing are evaluated and summarized in the geological model. In particular, the design values for the soil shear strength parameters are usually determined using triaxial testing results of undisturbed soil samples obtained from bore holes and trial pits. The design ground water table is usually assumed according to the results of piezometer monitoring, wetting band theory, and/or site observations. The factual data on the ground condition are always too limited to allow a geotechnical designer to have a full understanding of the ground conditions. Certain assumptions and geotechnical judgments have to be made in the process of geological model development. The adopted geological model is usually a deterministic representation of the ground conditions. Uncertainties and variations in the ground conditions may not be fully represented by the geological model.

After the construction of the geological model, the subsequent tasks are to carry out slope stability assessment and soil nail design calculations. At first, the factor of safety (FOS) of existing slope is calculated using the conventional slice methods such as the simplified Janbu method. Potential slip surfaces with FOS values less than a predetermined value (say, 1.2 or 1.4) are considered to be substandard and to be stabilized with soil nails. Secondly, horizontal supporting forces are applied on the slope surface to bring the FOS values of all the substandard potential slip surfaces up to the predetermined value. Calculations are carried out to determine the minimum horizontal supporting force that upgrades all the substandard potential slip surfaces with new FOS values greater than the pre-determined value. The minimum horizontal supporting force is assumed to be equal to the required pullout resistance of the soil nails. The third step is to determine the vertical

and horizontal spacing, lengths and steel bar diameters of soil nails to meet the following criteria: (a) the total available pullout resistance of the soil nails should be greater than the required pullout resistance; (b) the maximum pullout resistance of each soil nail should not be greater than both the allowable tensile strength of each steel bar and the allowable bond resistance between the steel bar and the grout. The partial factors of safety are usually adopted in checking pullout resistance, bar tensile strength and corrosion protection.

Soil Nail Construction & Testing

The construction of a soil nail follows the following four steps: a) drill a hole of diameter 100 or 150 mm and at a declination angle 10 to 30° to horizontal in the ground; b) insert the slender steel bar into the pre-drilled hole; c) grout the open space between the bar and the surrounding ground in the hole; d) construct a nail head. Each of the four steps is usually applied to a group of soil nails for one construction stage. The soil nail is designed to have sufficient tensile strength to resist forces induced by any lateral displacements within the slope. Pullout tests are required to verify the pullout resistance adopted in the soil nail design. Such pullout resistance is controlled by the three failure modes in association with the design criteria. In particular, the bond strength between the grout and the surrounding soils has always played a key role in the design. As a result, the substandard slope is reinforced and has an increased overall safety up to the current safety standard. Such practice in Hong Kong is consistent with those used in Mainland China and many other countries.

Key Issue in Current Soil Nailing Practice

As discussed above, the adopted geological model is usually simple and constructed according to limited factual data. Spatial variations in the ground conditions may not be fully represented by the geological model. In particular, the soils comprising slopes in Hong Kong are mainly weathered rock soils. Characteristics of the weathered rocks have been well examined and understood (Lumb 1980; GEO 1988; Fyfe 2000; Martin 2003). One of the basic characteristics is that “Weathered rock profiles are often complex and irregular. Variability cannot usually be predicted using standard methods of geological interpolation and extrapolation” (Hencher and McNicholl 1995). Consequently, it is highly possible that the current soil nailing practice can result in either under or over specifications of soil nail lengths. For a slope that is stabilized with soil nails of under specification, it can still be potentially danger to the public. For a slope that is stabilized with soil nails of over specification, its construction is definitely not cost-effective. Therefore, there has been a pressing need to address this key issue in current soil nail practice. It is against such background that the DPM method was developed to resolve the problem by accurately and effectively determining the pullout resistance between the grout and the surrounding weathered rock/soils for each soil nail.

DPM FOR GEOMATERIAL STRENGTH DISTRIBUTION ALONG NAIL HOLE

There are at least three types of drilling works associated with the soil nailing for slope stabilization. The first drilling work is to drill holes in the ground for the ground investigation including in-situ testing and sampling. The second is to drill holes for installation of soil nails for pullout testing. The third is to drill holes for installation of permanent soil nails in the slopes. Consequently, many holes are drilled in the ground. The drilling work for ground investigation is reported in drillhole logs. The drillhole logs are usually prepared by engineering geologist according to the soils and rocks recovered from the investigation holes. The drilling work for soil nail installation is usually documented as

record sheets. The record sheets are usually prepared by the operator after the drilling work is completed and according to the operator's experience and judgment.

It has been well recognized that drilling progress in the ground is related to the mechanical strength of the ground geomaterials. We have found that drilling a hole in the ground itself can be used as an in-situ test to measure the mechanical strength of the geomaterials surrounding the hole. Such material strength is the geomaterial resistance to drilling and is related to the geomaterial pullout resistance for soil nails. Such drilling resistance of the geomaterials in a hole can be measured via automatically and continuously monitoring the drilling process while the hole is being drilled in the ground. So far, we have monitored the drilling processes for up to 500 soil nail holes at more than 10 LPM construction sites. The lengths of the soil nail holes are between 4 and 40 m. We have analysed the monitored DPM data and discovered that such factual data in time series for a nail hole can be used to measure the drilling resistance distribution in depth along the nail hole. In the ensuing, we will discuss this DPM method for characterizing the mechanical strength with the depth along the nail hole.

Drilling Machines

There are many different drilling machines for different applications and objectives in geotechnical engineering and construction (Chugh 1985; GEO 1987). They include the drilling machines for ground investigation, bored piles, driven piles, soil nailing, raking drains, rock bolts and ground anchors. There are four basic drilling methods: (a) auger boring, (b) rotary drilling (or boring), (c) percussive drilling (boring or driving), and (e) percussive rotary drilling (or boring). Hydraulic power may be used for the auger boring and rotary drilling. Either hydraulic or pneumatic power may be used for the percussive drilling and rotary-percussive drilling. The drilling machines may be and/or may not be able to core or recover soil or rock samples from the hole in the ground.

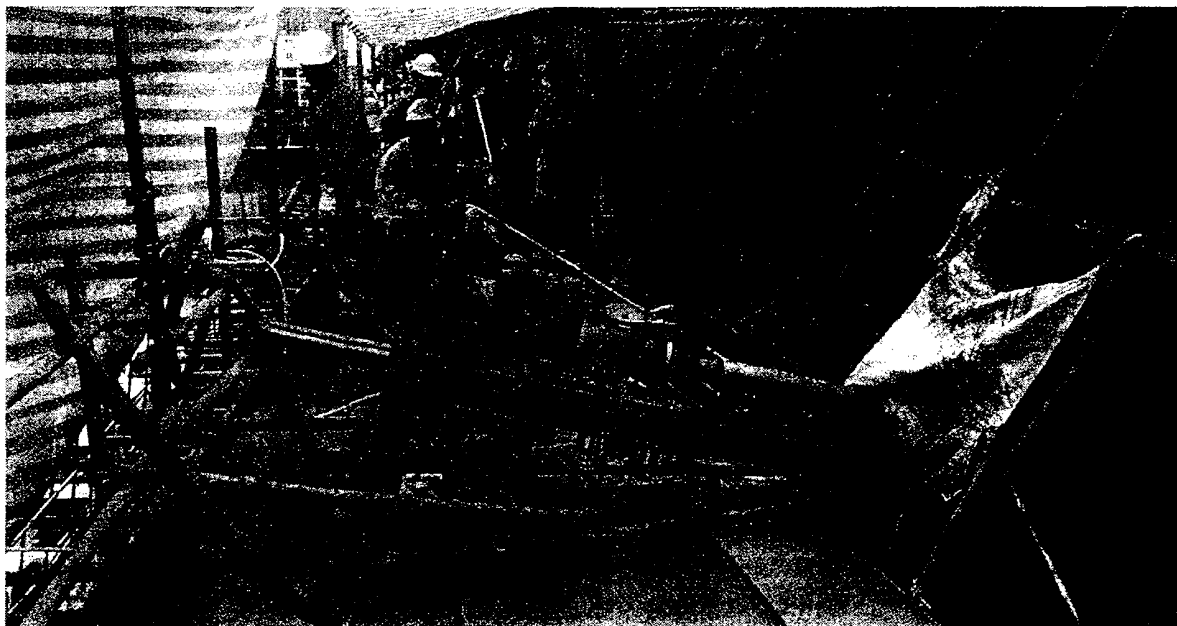


Figure 1 Pneumatic rotary-percussive drilling machine equipped with DPM in drilling a soil nail hole in a cut slope at Ho Man Tin, Kowloon, Hong Kong

The pneumatic rotary-percussive drilling machines equipped with down-the-hole hammer are commonly used in drilling the holes for installation of soil nails in Hong Kong. Figure 1 shows the typical examples of the drilling machines. This drilling machine consists

of a down-the-hole hammer with drill bit, extension rods, swivel chuck, two feed gears, an oil lubricator, a chain feed, two motors, a sliding beam and a control panel. It is operated with pneumatic power from a compressor in the form of compressed airflow in a main hose.

The drilling machine penetrates the ground mainly by the percussion action with the assistance of the rotational action and the thrust action of the drilling bit. The percussive action shatters the soils and rocks in the ground and the rotational action tears them. A main compressor supplies the compressed air to the control panel. The control panel is operated manually and separated the compressed air into five different hoses to control the drilling. Two hoses supply the compressed airflows to the swivel chuck for forward and backward rotating the drill rods and bit. One hose supplies the compressed airflow into the down-the-hole hammer for percussion and flushing. The other two hoses supply the compressed airflows to the thrust motor to pull down or up the swivel chuck via the chain feed. The swivel chuck slides up or down on the sliding beam.

Such drilling machines have the following features: a) wide drilling versatility, for example, drilling direction can be vertical, horizontal or at any inclined or declined angles; b) high drilling efficiency in a wide range of geomaterials including weathered granite and tuff; c) low noise generation; and d) various site accessibility due to their light weights and small sizes. They provide a fast, powerful and economical tool that can drill holes in both soils and rocks and on high and steep slopes. The drilling work is usually carried out on steel scaffolding platforms.

HKU Drilling Process Monitor

In the light of the popularity of soil nailing technique as a slope stabilization measure in Hong Kong and around the world, we have developed an in-situ device, named as the HKU Drilling Process Monitor (DPM) to automatically measure and record the drilling process of the drilling machine in a digital manner and in real time. The drilling process is defined by the drilling parameters in time series. The parameters include the translational and rotational movements of the swivel chuck and the five compressed airflow pressures that power the movements. Figure 2 shows the design sketch of the DPM.

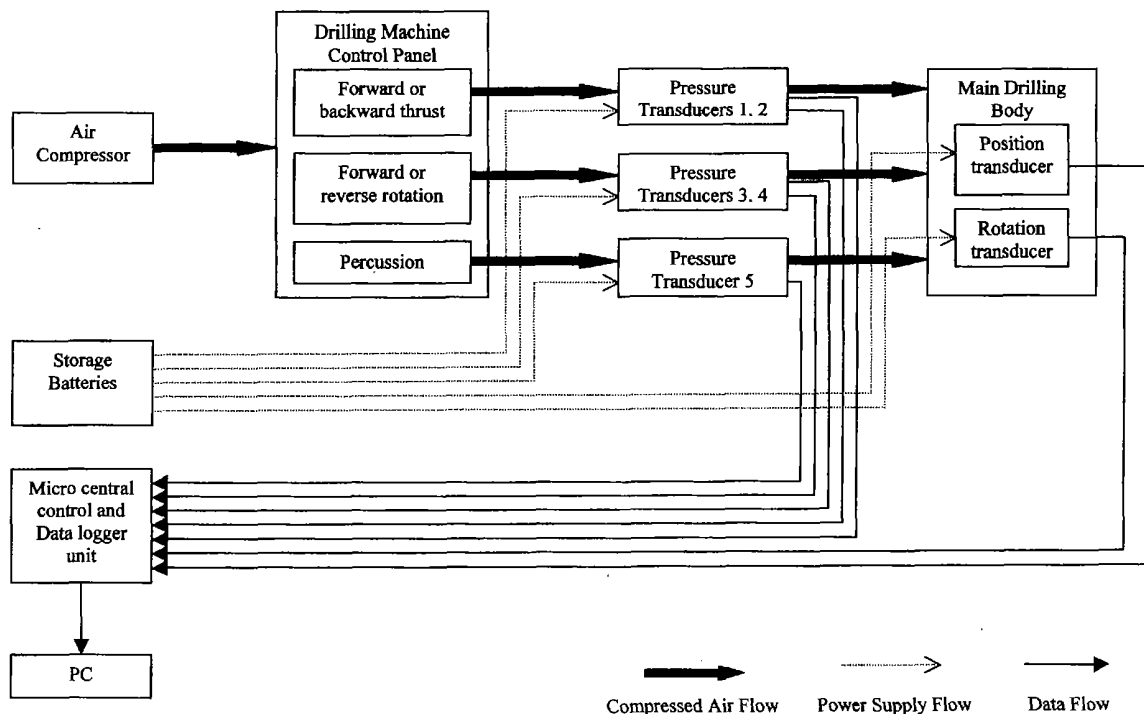


Figure 2 Design sketch of the HKU Drilling Process Monitor

The DPM comprises a sensor unit and a data processing unit. The sensor unit includes seven transducers to measure the physical drilling parameters. A position transducer measures the translational movement of the swivel drill chuck on the sliding beam. A rotational transducer measures the rotation speed of the drill rods and bit. Five pressure transducers measure the five compressed airflow pressures for the thrust, rotational and percussive actions. The data processing unit includes a micro central control unit and a portable data saving unit. The micro central control unit simultaneously acquires the output signals from the two movement transducers and the five pressure transducers in the form of voltage output or electrical pulses. The output signals are transmitted to a portable unit of the data acquisition system at a selected sampling frequency (e.g., 1 to 10 times per second) and then stored and plotted in a personal computer.

The sensor unit and the data processing unit are small in size and are portable. Additional sensors can be easily added to the system. Mounting the sensors on the machine or removing the sensors from the machine are an easy and simple task and can be completed within a short time. The mounting and removing work have no adverse effect to the drilling machine (Figure 1). All sensors are heavy duty and are robust to endure various site conditions including dust, water, heat, light, and vibrations. A battery is used to supply a small amount of power for the device. As a result, the DPM is versatile, simple and can accommodate any rotary-percussive drilling machines and other drilling machines with minor modifications.

DPM Data

In the ensuing, we will focus on the analysis of the test data obtained from a slope stabilization project in Hong Kong. The original data were monitored and recorded automatically with the DPM during the process of drilling a soil nail hole in real time. The selected cut slope was formed 20 years ago and had experienced instability. Its height is 35 m. The cut slope comprises weathered tuff of Jurassic to Cretaceous age.

Figures 3 to 8 show the original DPM data for the complete process of the drilling with a pneumatic rotary-percussive drilling machine for a soil nail hole in the cut slope. Such complete drilling process data includes the drilling preparation at the very beginning, the drilling progress into the hole and the last operation for retrieving the extension rods from the hole. Figure 3 shows the movement of the swivel drill chuck with respect to a fixed point on the sliding beam with respect to the actual drilling time. Figure 4 shows the complete time history of the application of the compressed airflow pressure to the down-the-hole hammer for percussion. Figure 5 shows the complete time history of the application of the compressed airflow pressure to the rotation motor for forward rotation action of the drill rod and bit. Figure 6 shows the complete time history of the application of the compressed airflow pressure to the rotation motor for reverse rotation action of the drill rod and bit. Figure 7 shows the complete time history of the application of the compressed airflow pressure to the thrust motor for forward translational movement of the drill rod and bit. Figure 8 shows the complete time history of the application of the compressed airflow pressure to the thrust motor for upward translational movement of the drill rod and bit.

From the data in Figures 3 to 8, we can have the following observations on the drilling operations. It used ten extension rods and one down-the-hole hammer. Each rod was about 3 m long. The hole depth was about 33 m. The preparation time was from 12:33:08 to 12:39:33, when the hammer was added. The downward drilling operation started at about 12:39:33 and completed at about 14:41:53. The ten extension rods were added and drilled into the ground. Retrieving of the rods then started at about 14:42:53 and completed at about 14:58:14.

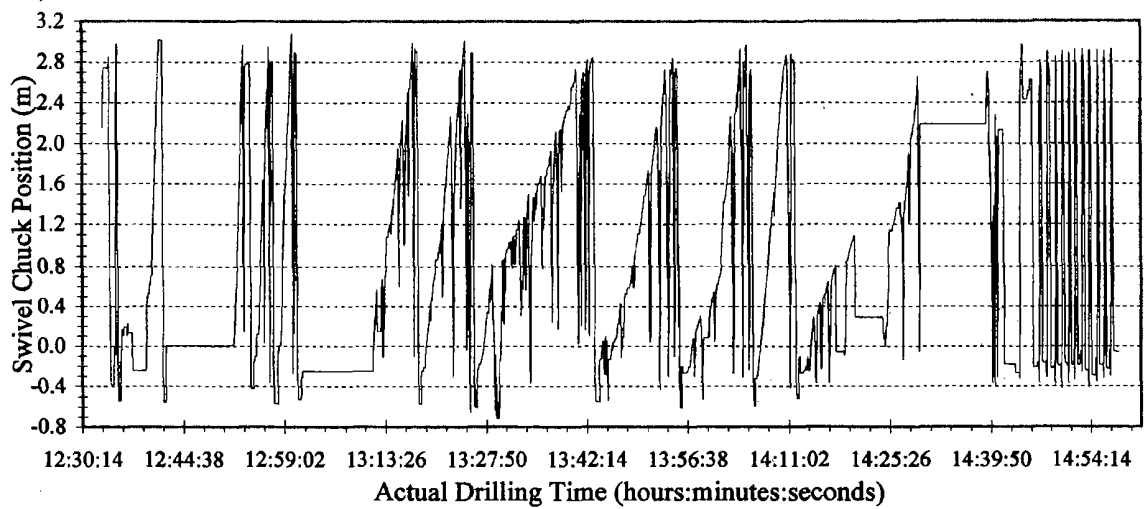


Figure 3 Translational movement of the drill chuck with actual time for drilling a soil nail hole

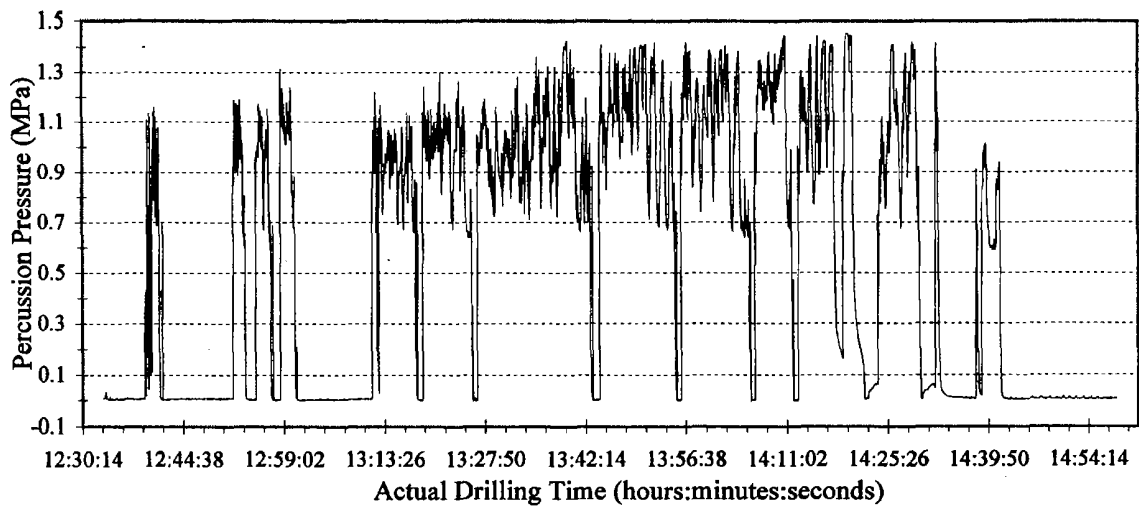


Figure 4 Variation of the percussion pressure with actual time for drilling a soil nail hole

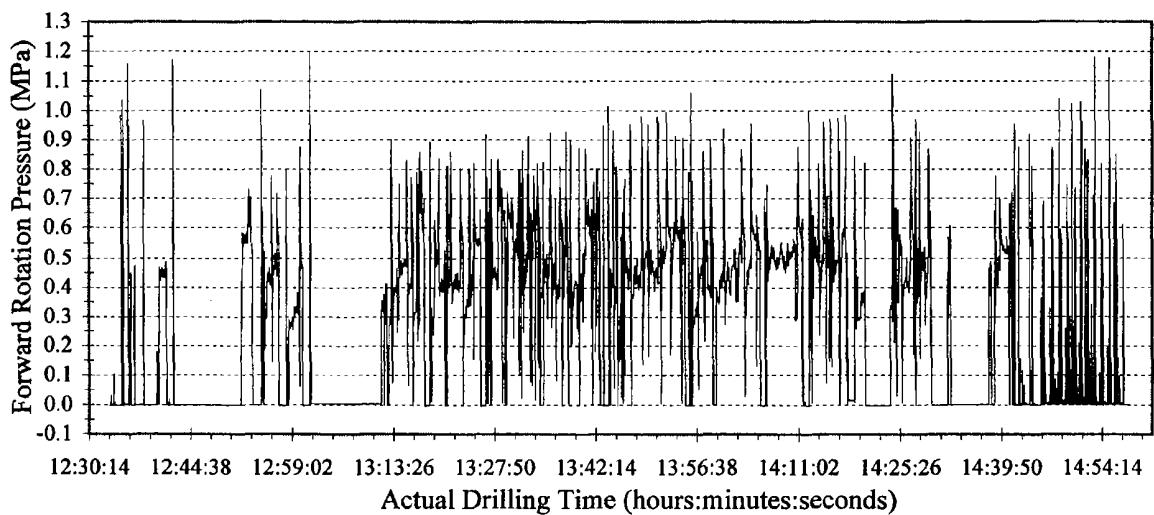


Figure 5 Variation of the forward rotation pressure with actual time for drilling a soil nail hole

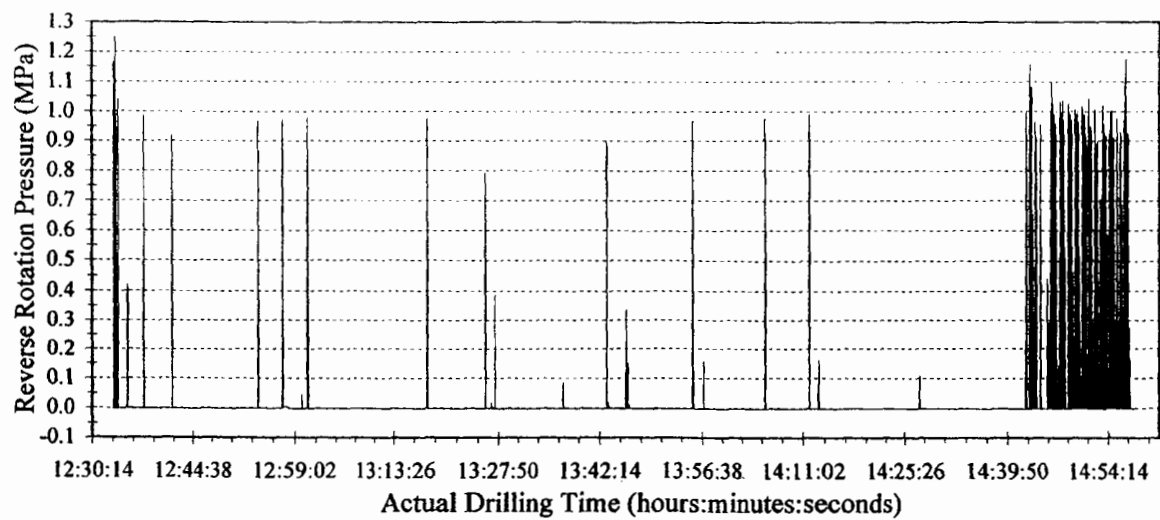


Figure 6 Variation of the reverse rotation pressure with actual time for drilling a soil nail hole

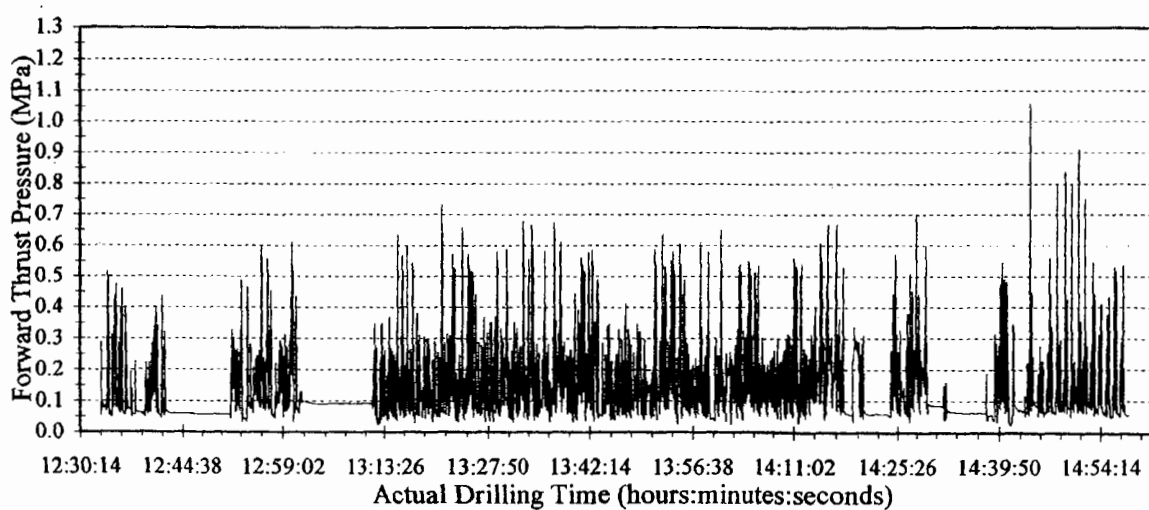


Figure 7 Variation of the forward thrust pressure with actual time for drilling a soil nail hole

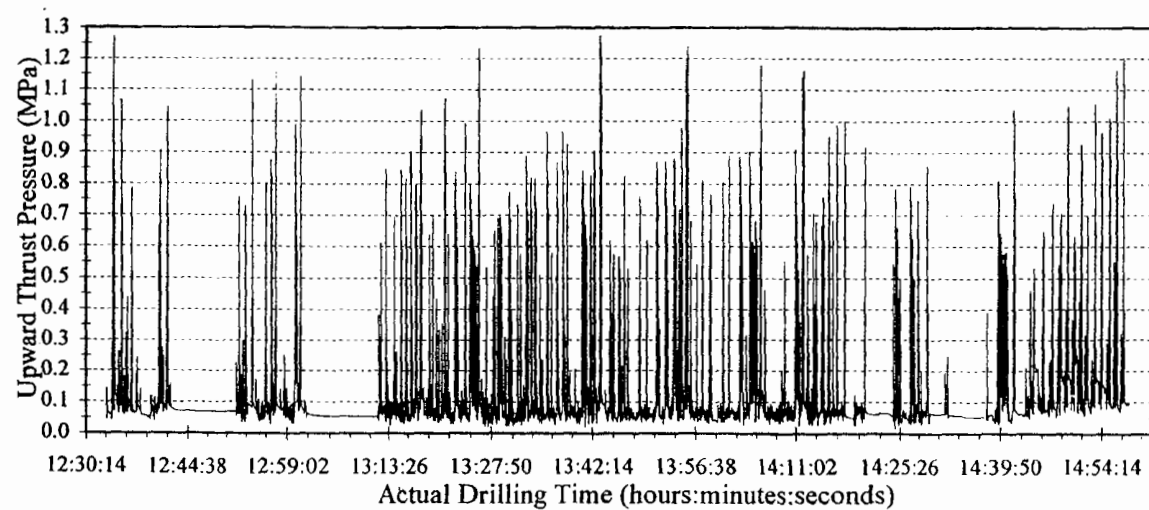


Figure 8 Variation of the forward thrust pressure with actual time for drilling a soil nail hole

A detailed examination of the recorded drilling parameters in Figures 3 to 8 has shown that the drilling process can be classified into the following seven distinctive processes:

- Penetrating process, which stands for the advancement of drill hammer bit into new geomaterial in deeper depth;
- Pushing in or pulling back processes, which stand for the downward or upward movements of the chuck and rods without actual drilling of the geomaterial in the hole and without application of percussion air pressure, respectively.
- Flushing processing, which stands for the downward or upward movements of the chuck and rods without actual drilling of the geomaterial in the hole and with application of percussion air pressure.
- Tightening or loosening processes, which stand for the connection or disconnection of one rod segment with the existing rods in the hole.
- Stoppage process, which stands for the period when all the compressed air pressures are not being supplied.

We have written computer programs to identify and distinguish the above seven drilling sub-process. In particular, the hammer bit advancement with the net time associated purely with the penetrating process can be used to estimate the drilling rate of the down-the-hole hammer bit in the ground. For the DPM data in Figures 3 to 8, the hammer bit advancement with the net drilling time is shown in Figure 9. It is evident that the gradient of the curve in Figure 9 is the drilling rate of the hammer bit for drilling the soil nail hole.

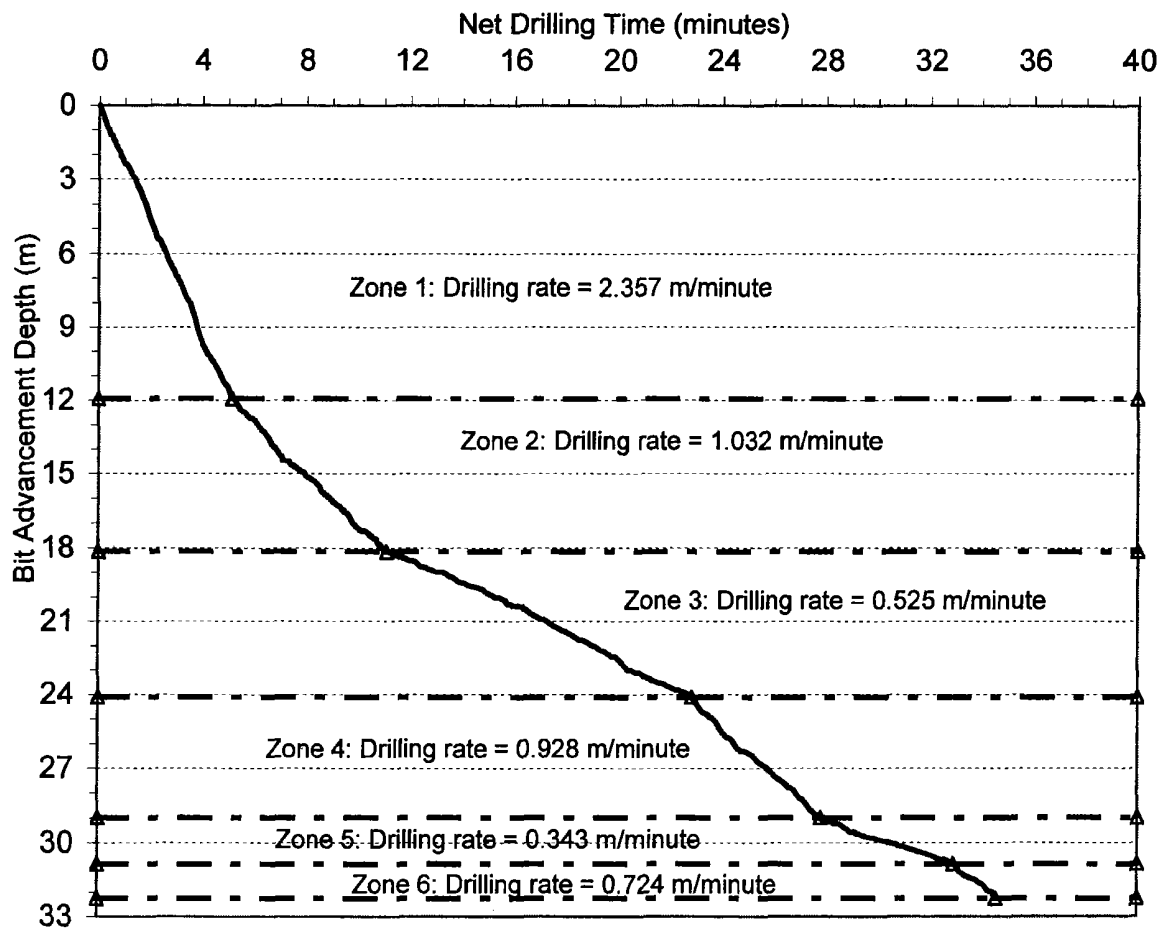


Figure 9 Depth advancement of the drill bit in a cut slope with the net drilling time and DPM zoning result along the hole

From Figure 9, we can find that the hammer bit advancement with the net drilling time can be distinctively classified into six linear variation zones. The gradient value of this curve is a constant in each linear zone. In other words, the drilling rate of each linear zone is a constant. The detailed gradient values of the six linear zones are shown below:

Zone 1: which is from the surface to the depth 11.94 m and its average drilling rate is 2.357 m/minute.

Zone 2: which is from the depth 11.94 m to the depth 18.16 m and its average drilling rate is 1.032 m/minute.

Zone 3: which is from the depth 18.16 m to the depth 24.12 m and its average drilling rate is 0.525 m/minute.

Zone 4: which is from the depth 24.12 m to the depth 28.99 m and its average drilling rate is 0.928 m/minute.

Zone 5: which is from the depth 28.99 m to the depth 30.86 m and its average drilling rate is 0.343 m/minute.

Zone 6: which is from the depth 30.86 m to the depth 32.26 m and its average drilling rate is 0.724 m/minute.

Geomaterial Strength along Hole Depth

The drilling rate is a signature for the geomaterial resistance strength against drilling or damage. The lower the penetration rate, the stronger the ground material, and vice versa. For weathered tuff, the higher the decomposition, the lower the tuff strength and the higher the drilling rate under similar external drilling force or energy.

From Figure 9, we can observe that the drilling rate varies with depth along the hole. The drilling rate can be classified into six constant rates representing six geomaterial zones. The constant drilling rates associated with any two connected geomaterial zones are substantially different. One may ask why the variation of the drilling rate with depth can be divided into six constant segments and what caused such phenomenon?

Based on the functions of the drilling machine and the distribution of weathered tuff profile, we have found that such result is mainly due to the variations of the physical and mechanical characteristics of the geomaterial with the depth along drill hole. Each constant drilling rate zone represents a relatively homogeneous zone of the geomaterials. Therefore, the geomaterial resistance to drilling varies with depth in the manner of a piecewisely stepped constant function.

The above finding can be validated through analyzing the driving pressures of drilling machine. From Figure 4 to 8, we can observe that the variations of the three pressures for percussion, forward rotation and forward thrust actions with the actual time do not follow a uniform pattern for each of the six zones. Such variations of the three driving forces would not be able to be the cause of the constant drilling rates in the six linear zones. Furthermore, we can estimate the average compressed airflow pressures for the three driving forces for each of the six DPM constant drilling rate zones. They represent the applied energy level for the drilling of each unity depth in the hole. The results are shown in Figure 10. From Figure 10, we can observe that the average values of the three driving pressures change slightly for the different constant zone drilling rates. In particular, the lower drilling rates correspond to higher average values of the three driving pressures. Such results further confirm that the constant distribution of the drilling rate in each DPM zone is due to the geomaterial resistance to the drilling.

In order to take into account the effect of the three driving forces, we define the following normalized drilling rate for a DPM zone in a soil nail hole as follow:

Normalized drilling rate = Measured drilling rate $\times$ Normalized driving pressure factor
(1)

where the normalized driving pressure factor $A_{zone I}$ for the I-th zone can be defined as the following two forms.

$$A_{zone I} = \frac{\sum_{k=1}^K (\bar{p}_{zone k-P} \times \bar{p}_{zone k-T} \times \bar{p}_{zone k-R})}{K(\bar{p}_{zone I-P} \times \bar{p}_{zone I-T} \times \bar{p}_{zone I-R})} \quad (2)$$

or

$$A_{zone I} = \frac{\sum_{k=1}^K (\bar{p}_{zone k-P} + \bar{p}_{zone k-T} + \bar{p}_{zone k-R})}{K(\bar{p}_{zone I-P} + \bar{p}_{zone I-T} + \bar{p}_{zone I-R})} \quad (3)$$

where K is the total number of the identified constant drilling rate zones; $\bar{p}_{zone k-P}$, $\bar{p}_{zone k-T}$ and $\bar{p}_{zone k-R}$ are the average values of the three driving pressures for percussion, downward thrust and forward rotation actions for the zone k , respectively ($k = 1, 2, \dots, K$ and $1 \leq I \leq K$).

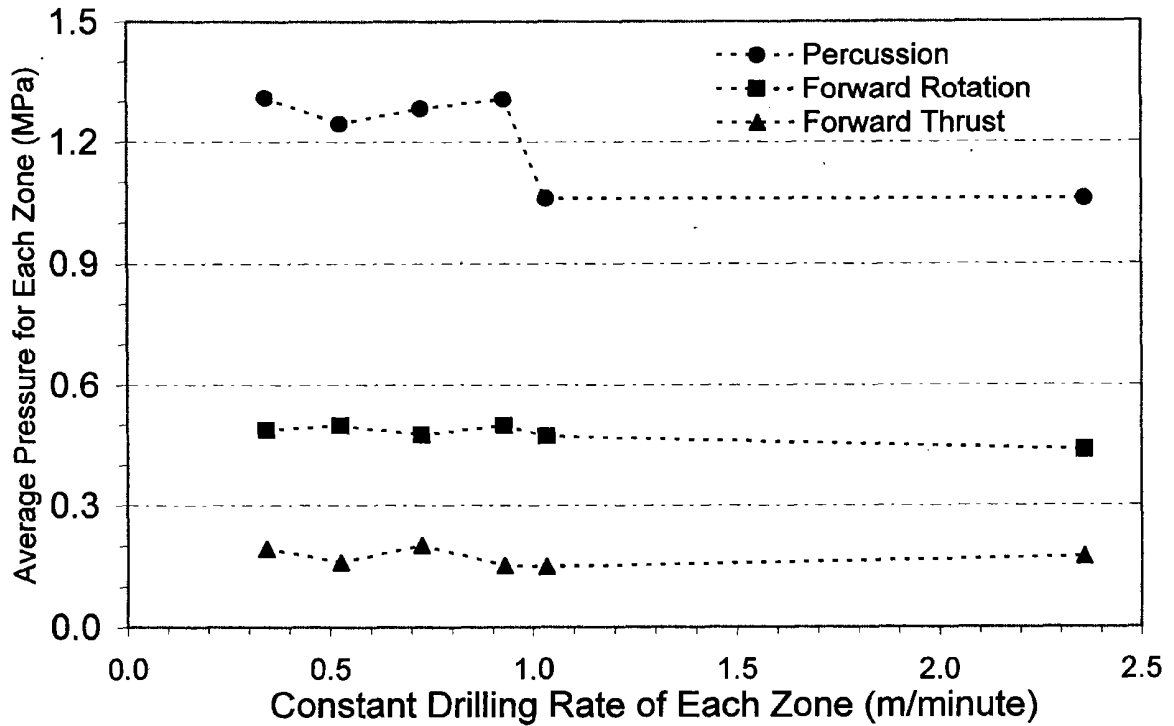


Figure 10 Average values of the three driving pressures for each DPM zone v.s. the constant drilling rate for the corresponding DPM zone.

We can further define the following parameter as the geomaterial resistance to drilling $D_{resistance}$ for each constant drilling rate zone.

$$D_{resistance} = \frac{1}{\text{Normalized drilling rate}} \quad (4)$$

According to the results for 500 m drilling depth of weathered tuff, we have found that

the $D_{\text{resistance}}$ is between 20 to 0.125 minutes per meter penetration for the complete spectrum of the weathered tuff profile from fresh tuff, slightly decomposed tuff, moderately decomposed tuff, highly decomposed tuff, completely decomposed tuff and residual soils (Yue et al. 2004).

Based on our studies, we have the following geological interpretations on the DPM results in Figures 9 and 10: Zone 1 is a completely decomposed tuff (which was also exposed on the slope surface); Zone 2 is a highly decomposed tuff; Zone 3 is a moderately decomposed tuff; Zone 4 is a highly decomposed tuff; Zone 5 is a moderately decomposed tuff; and Zone 6 is a highly to moderately decomposed tuff. Since there could be factors other than types of soil and rock which could affect drilling rate, more experience would be needed to correlate the types of soil/rock strata and the drilling rate based on actual borehole data. One of the other factors is the possible presence of groundwater during drilling a deep depth in the ground. The drilling rate could be affected if water is present as a drilling medium.

DPM METHOD FOR RATIONALIZING SOIL NAILING

As discussed above, the mechanical strength of the geomaterial in a soil nail hole plays an important role in providing the necessary pullout resistance against the soil nail pullout failure. The mechanical strength of the geomaterials in the pullout resistance zone must be adequately high. Hence, the above DPM method for identifying the mechanical strength of geomaterial along each soil nail hole can be used to rationalize the design and construction of soil nail length. We consider that if a geomaterial has a high resistance against drilling, it would have a high resistance against the pullout failure of a soil nail in such geomaterial, and *vice versa*.

Pullout Resistance

The available pullout resistance for a soil nail embedded in soil T_{soil} can be estimated using the following equations (Powell and Watkins 1990; Watkins and Powell 1990)

$$T_{\text{soil}} = \pi DL \left(c' + \frac{2}{\pi} \sigma'_z \tan(\phi') \right) / \text{FOS} \quad (5)$$

where D is the diameter of soil nail, L is the bonded nail length, c' is the soil effective cohesion, ϕ' is the soil effective frictional angle, σ'_z is the effective vertical stress (overburden pressure) at the centre of the bonded nail length in the stable soil zone, and FOS is a partial factor of safety and usually is 2.

Figure 11 shows the variation of the available pullout resistance intensity $T_{\text{soil}} / \pi DL (= (c' + \frac{2}{\pi} \sigma'_z \tan \phi') / \text{FOS})$ with respect to the effective vertical stress σ'_z at the upper and lower bounds of effective shear strength parameters for completely decomposed rock soils in Hong Kong.

On the other hand, the available pullout resistance for soil nail embedded in sound rock (grade III or better) T_{rock} can be estimated using the following equation.

$$T_{\text{rock}} = \pi DL \tau_{\text{available}} \quad (6)$$

where $\tau_{\text{available}}$ is the available skin frictional strength between grout and sound rock and is assumed to be 200 kPa as shown in Figure 11.

From Figure 11, one can estimate that the available pullout resistance intensity $\tau_{\text{available}}$

for soil nail embedded in sound rock is much about 1.8 to 17.5 and 1.3 to 10.1 times greater than those of the lower and upper bound values of the available pullout resistance intensity for the completely decomposed soils. Since the degree of the weathering of the rock mass varies continuously from completely weathered (100% weathering) to unweathered (0% weathering) (GEO, 1988), there are intermediate weathered geomaterials whose available pullout resistance intensities are between those of the sound rocks and the lower bound of the completely decomposed soils. In other words, the soil nail length can be substantially shortened if a stronger ground geomaterial condition is encountered, and *vice versa*.

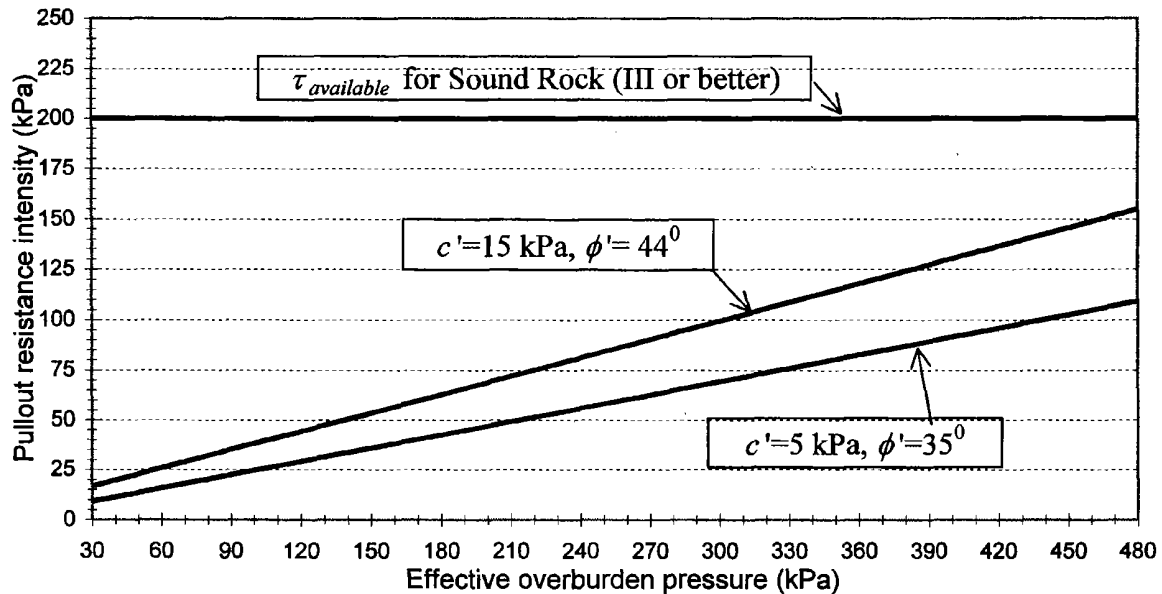


Figure 11 Available pullout resistance intensity for completely decomposed soils v.s. effective overburden pressure (equivalent to the buried depth of 2 to 30 m).

Such concept has been adopted in the stipulation in the prescriptive design method that “If in the process of drilling rock is encountered such that part of the soil nails will be installed in rock (e.g., installation through a PW50/90 zone or better, see Geoguide 3), the designer may exercise professional judgement or to reduce the nail length L . In the absence of site-specific ground investigation, sound engineering judgement is required to ensure that the nails are not prematurely terminated in corestone boulders” (Wong and Pang, 1999). It is therefore cost-effective and independent to use the DPM method in identifying the variation of the geomaterial strength with depth along a soil nail hole. The results shown in Figures 8 and 9 clearly demonstrate that the geomaterial strength varies and gets higher as the depth increases.

Application Example

Another example of the DPM application to a cut slope stabilization work is given below to show the above concept. The cut slope immediately after the completion of soil nailing and hydroseeding is shown in Figure 12. The cut slope was formed about thirty year ago by cutting into a hillside for housing development. The cut slope is about 40 m high and has an overall slope angle of 50°. The cut slope is underlain by medium grained granite of Jurassic to Cretaceous age. The completely decomposed granite and a number of large core-stones are exposed on the cut slope surface. The core-stones are mainly slightly decomposed granite.

Figure 13 shows the bit advancement with the net drilling time. It is evident that the curve consists of five linear segment lines. Each linear segment line has a constant gradient

for the drilling rate in this homogeneous geomaterial zone. They can be described as follows.

- Zone 1: which is from the slope surface (0.0 m) to the depth 4.44 m and its average drilling rate is 1.78 meter per minute.
- Zone 2: which is from the depth 4.44 m to the depth 5.85 m and its average drilling rate is 0.10 meter per minute.
- Zone 3: which is from the depth 5.85 m to the depth 8.71 m and its average drilling rate is 1.85 meter per minute.
- Zone 4: which is from the depth 8.71 m to the depth 10.02 m and its average penetration rate is 0.10 meter per minute.
- Zone 5: which is from the depth 10.02 m to the hole end 20.91 m and its average penetration rate is 0.72 meter per minute.



Figure 12 A cut slope after stabilization with soil nails and hydroseeding in Hong Kong

Based on the completely decomposed granite and the core-stones exposed on the cut slope surface, Zones 1 to 5 can be further interpreted as a completely decomposed granite soil zone, a slightly decomposed granite core-stone zone, another completely decomposed granite soil zone, another slight decomposed granite core-stone zone, and a highly to moderately decomposed granite rock zone, respectively.

Furthermore, the original geological model used for the soil nail design is shown in Figure 14 below. The above DPM result in Figure 13 is for one of the soil nails in the second row from the crest. We can draw the above DPM zoning results on the geological model along the second soil nail row. It is evident that the geomaterial along the row 2 behind the potential slip surface No.3 was found to be a highly to moderately decomposed granite. In the original design model, it was assumed to be a completely decompose granite soil that is

exactly the same as that exposed on the cut slope surface. Besides, two corestones of the measured lengths 1.41 and 1.31 m were found in the completely decomposed granite soil above the potential slip surface No.3. It is clear that the soil nail design scheme might be over specified. The CDG soil between the two corestones could cause stability problem if the soil nail was terminated in the first corestone at depth between 4.44 and 5.85 m.

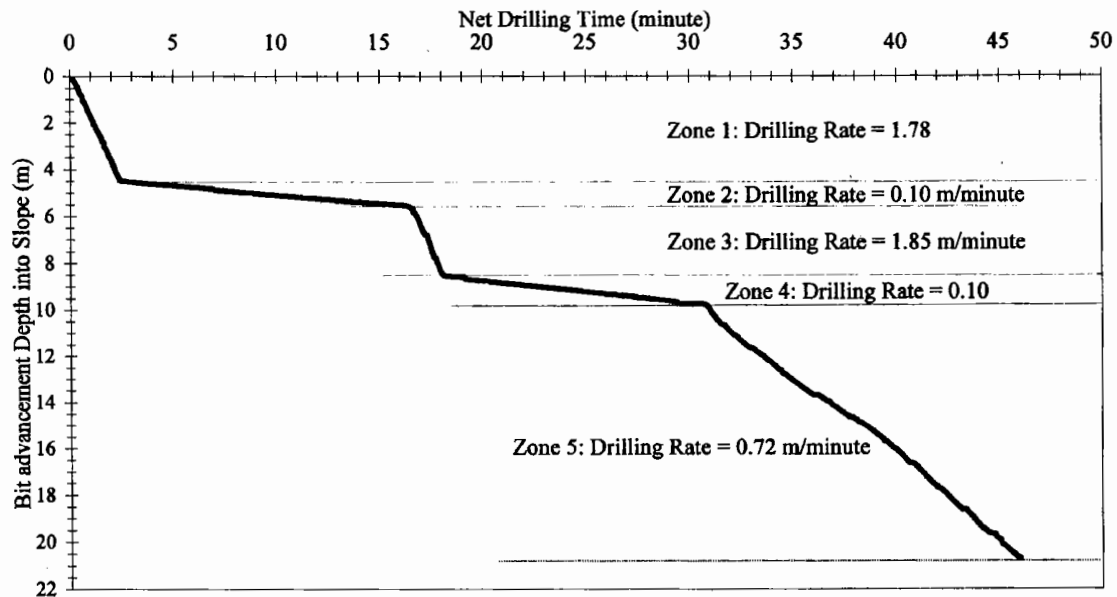


Figure 13 Depth advancement of the drill bit in the cut slope in Figure 12 with the net drilling time and DPM zoning result along the hole

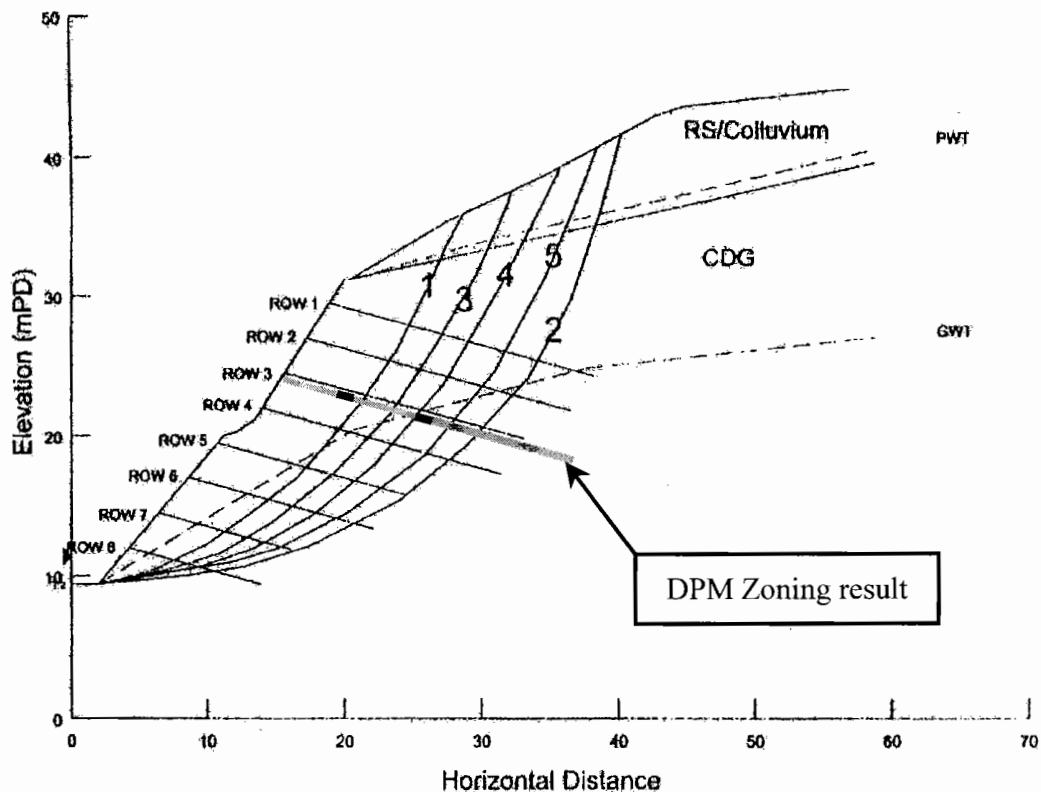


Figure 14 Original geological section for soil nail design of a cut slope and DPM zoning result along a soil nail at the row 2

Procedure for Rationalization

The cost for soil nailing includes the works for ground investigation, scaffolding, plant mobilization, drilling holes, pullout tests, steel bars, grouting, and nail head construction. Among these works, the rationalization with the DPM method could cover the works for ground investigation, drilling holes, pullout tests, steel bars and grouting. The following is one of the procedures that may be adopted in rationalizing soil nail design.

At present, hydraulic rotary drilling of boreholes for sampling and borehole testing is commonly used in analytically based soil nail design methods. So, the pattern of the geomaterial resistance to rotary drilling at a slope site should be established with the DPM method during ground investigation.

Besides, pullout tests are normally carried out during construction stage for checking and verification of the soil nail design results. The soil nail holes can be carried out quickly with the rotary-percussive drilling technique. Such pullout tests could be carried out during the ground investigation stage for better soil nail design scheme. Their holes drilling with the rotary-percussive machines could be monitored with the DPM method. In particular, the DPM results could also be used to determine the 2 m bond zone with known drilling resistance for each pullout test.

The DPM results from the pullout test holes could further be used for verification of the soil nail design scheme during the construction stage, when sufficient experience has been accumulated for the use of DPM. The drilling machines used for drilling permanent soil nail holes should be similar to the drilling machines for the pullout soil nail holes. Then some guidance could be developed for verification of the DPM results from the automatic drilling process monitoring of permanent soil nail holes in real time. The guidance could include the criteria for termination or elongation of the drilling depth on site.

It is noted that the hydraulic rotary drilling for borehole exploration and the pneumatic rotary-percussive drilling for soil nail installation have substantially different mechanisms in penetrating into the geomaterials (GEO, 1987). Their drilling rates for same geomaterials can also be substantially different.

CONCLUSIONS

This paper has presented the use of a digital drilling process monitor (DPM) in characterizing the profile and mechanical strength of weathered geomaterials during drilling production holes for soil nail installation. A possible method to further rationalize the design and construction of soil nails in Hong Kong through using the DPM method is proposed in this paper. The two case studies presented above have shown that the DPM data could provide information on the relative drilling resistances of the geomaterials surrounding each soil nail in the ground.

The DPM method could further promote the recognition and practice of soil nailing techniques for slope stabilizations in Hong Kong and around the world. DPM experience and data will be gradually gained and accumulated for routine and wide application in geotechnical design and construction. We believe that similar to the conventional cone penetration test method, the DPM method could, once sufficient experience has been gained, become a common and powerful test method for continuous measurement of the geomaterial mechanical strength. It could be applicable to a much wider range of geomaterials including soils, soft rocks and hard rocks. The DPM data are factual digital data that have not been recorded and utilized before. They could be used for improvement of geotechnical investigation and design, geotechnical design verification during construction, construction management, as well as quality control of drilling works. The invention of the DPM method has resulted a great potential in furthering the geotechnical knowledge and engineering practices, in particular, the soil nailing for slope stabilization in hilly terrain regions.

REFERENCES

- Campanella, R. G., Robertson, P. K., Gillespie, D. G., and Greig, J. (1985). "Recent developments in in-situ testing of soils." in *Proceedings of 11th Int. Conf. Soil Mech. Foundation Engg.*, San Francisco, USA, Vol. 2, pp.849-854.
- Chung, K. F., Yong, Y. K., and Tsui, H. M. (1992). "Ground support using soil nails: some Hong Kong case histories." In *Proceedings of Seminar on Lateral Ground Support Systems*, Geotechnical Division, Hong Kong Institute of Engineers, Hong Kong, May, p.105-121.
- Chugh, C. P. (1992), *High Technology in Drilling and Exploration*, A.A. Balkema/Rotterdam, VT 05036, U.S.A.
- Fyfe, J. A., Shaw, R., Campbell, S. D. G., Lai, K. W., and Kirk, P. A. (2000). *The Quaternary Geology of Hong Kong*. Geotechnical Engineering Office, HKSAR Government, Hong Kong.
- Gassler, G. (1990). "In situ techniques of reinforced soil." in *Proceedings of the International Conference on Performance of Reinforced Soil Structures*, British Geotechnical Society, Glasgow, UK, pp.185-196.
- GEO (1987), *Guide to Site Investigation, GEOGUIDE 2*, Geotechnical Engineering Office (GEO), Civil Engineering Department, HK Government, Hong Kong.
- GEO (1988). *Guide to Rock and Soil Descriptions, GEOGUIDE 3*. Geotechnical Engineering Office, Civil Engineering Department, Hong Kong Government.
- Hencher, S. R., and McNicholl, D. P. (1995). "Engineering in weathered rock." *Quarterly Journal of Engineering Geology*, 28: 253-366.
- Juran, I., and Elias, V. (1991). "Ground anchors and soil nails in retaining structures." In *Foundation Engineering Handbook*, edited by H. Y. Fang, Boston, pp.868-905.
- Lee, C. F., Law, K. T., Wang, S. J., and Yue, Z. Q. (1999). *Report on Research Priority and Strategy of the Jockey Club Research and Information Centre for Landslip Prevention and Land Development*, Department of Civil Engineering, The University of Hong Kong, Pokfulam Road, Hong Kong.
- Lumb, P. (1980). "Thirty years of soil engineering in Hong Kong. Rupert H. Myers Lecture, Leura." In *A Memorial Collection of Selected Papers and Memoir of Professor Peter Lumb*, Edited by A. T. Yeung, Professor Lumb Trust Fund, The University of Hong Kong, Pokfulam, Hong Kong, 2003. pp. 757-779.
- Martin, R. P. (2003). "Review of geological aspects of slope engineering." *Transactions of Hong Kong Institution of Engineers*, 10(3): 26-66, Hong Kong.
- Powell, G. E., and Watkins, A. T. 1990. "Improvement of marginally stable existing slopes by soil nailing in Hong Kong." In *Proceedings of the International Conference on Performance of Reinforced Soil Structures*, British Geotechnical Society, Glasgow, UK, pp.241-247.
- Sugawara, J., Yue, Z. Q., Tham, L. G., Law, K. T., and Lee, C. F. (2003a). "Weathered rock characterization using drilling parameters." *Canadian Geotechnical Journal*, 40(3): 661~668.
- Sugawara, J., Yue, Z. Q., Tham, L. G., Law, K. T., and Lee, C. F. (2003b). "Profiling weathered rock slopes using drilling parameters." *Proceedings of the International Conference on Slope Engineering*, December 8 to 10, 2003, Hong Kong, China, pp. 968-973.
- Wan, S. P., and Yue, Z. Q. (2003) "Effects of different slice methods on soil nail design for cut slope stabilization." *Proceedings of the International Conference on Slope Engineering*, December 8 to 10, 2003, Hong Kong, China, pp. 461-466.
- Wan, S. P., and Yue, Z. Q. (2004). "Significant cost implications of Janbu's simplified and Morgenstern-Price slice methods for soil nail design of cut slopes". *Transaction of*

- Hong Kong Institution of Engineers*, Hong Kong, 11(1): 54-63.
- Watkins, A. T., and Powell, G. E. (1992), "Soil nailing to existing slopes as landslip preventive works." *Hong Kong Engineer*, March, 1992, pp. 20-27.
- Wong, H. N. and Pang, L. S., (1999). *Application of Prescriptive Measures to Soil Cut Slopes*. GEO Report No. 56, Second Edition, CED, Government of HKSAR, Hong Kong.
- Yue, Z. Q., Lee, C. F., Law, K. T., Tham, L. G., and Sugawara, J. (2001). HKU-drilling process monitor and its application to slope stabilization, *Proceedings of the Conference on the Development and Co-operation of the Construction Industry of the Mainland and Hong Kong and Exploration of the International Market*, Works Bureau of HK SAR Government, Ministry of Construction of China, and The Hong Kong Institution of Engineers, Kunming, China, Vol. II, pp. II35~II46.
- Yue, Z. Q., Lee, C. F., Law, K. T., Tham, L. G., and Sugawara, J. (2002). "Use of HKU drilling process monitor in soil nailing in slope stabilization." *Chinese Journal of Rock Mechanics and Engineering*, 21(11): 1685-1690, Wuhan, China (in Chinese).
- Yue, Z. Q., Guo, J. Y., Tham, L. G., and Lee, C.F. (2003a). "Application of HKU DPM in automation of geotechnical design and construction." In *Proceedings of the 2003 Chinese National Congress on Geotechnics-Engineering*, Oct.22-25, 2003, Beijing; 1: 147-155. China Communications Press, ISBN 7-114-04724-X, China (in Chinese).
- Yue, Z. Q., Tham, L. G., and Lee, C. F. (2003b) "Parametric analysis of soil nail design for cut slopes." *Proceedings of the International Conference on Slope Engineering*, December 8 to 10, 2003, Hong Kong, China, pp. 554-560.
- Yue, Z. Q., Lee, C. F., Law, K. T., and Tham, L. G. (2004). "Automatic monitoring of rotary-percussive drilling for ground characterization - illustrated by a case example in Hong Kong." *International Journal of Rock Mechanics & Mining Science*, 41: 573~612.

ACKNOWLEDGEMENTS

The authors acknowledge the financial supports to this research by the Research Grant Council of HKSAR Government (No. HKU7005/01E) and the Hong Kong Jockey Club Charities Trust. The authors thank the kind supports from the Geotechnical Engineering Office, Housing Department, Halcrow China Ltd, Maunsell Geotechnical Services Ltd. and Fugro (Hong Kong) Limited. The comments from the Organizing Committee of the 2004 HKIE Geotechnical Division Annual Seminar are gratefully acknowledged.

AUTOMATIC MONITORING OF A SOIL NAILED LOOSE FILL SLOPE IN HONG KONG AND DATA ANALYSIS

J.-H. Yin¹ and X.L. Ding²

Abstract: This paper briefly describes an integrated automatic slope monitoring system and its application to monitoring slope movements. The system consists of an automatic conventional slope monitoring package and a multi-antenna Global Positioning System (GPS) package. This system was used to monitor a soil-nailed loose fill slope in Hong Kong. A large amount of field data was collected. In addition, numerical modelling of the slope movements has been carried out using FLAC. Field data obtained using the system and data from the FLAC modelling are presented, compared and interpreted. The assessment of the results has indicated that the integrated system is reliable, accurate and effective for monitoring slope movements and for warning of potential slope failures.

INTRODUCTION

Landslides have been a big concern of the Hong Kong community. Landslides have caused fatalities and great damage to property in Hong Kong. Monitoring potentially dangerous slopes is of practical and social significances. Landslides may be caused by heavy rain in summer, loading, and/or excavations. Before collapsing, slopes have certain pre-failure movements, large or small. Malone (1998) had an examination of pre-failure and reactivated movements of slopes after sliding in Hong Kong. It was found that deep-seated landslides moved normally slowly. Displacements before failure were from a few centimetres to about 1m or more.

Existing methods and technologies for slope monitoring have limitations. Traditional manual survey methods measure the movements of a slope surface only and are difficult to use when it is in heavy rain and/or the slope is in danger. Manually operated inclinometer measurement may measure the lateral movements of a slope. However, it is difficult to be used for a dangerous slope since operators are at risk when working on the slope. In this case, an automatic slope monitoring technique shall be used. In the past, automatic monitoring techniques, for example, using in-place inclinometers, have been developed for slope monitoring. In-place inclinometers can measure relative lateral slope movements accurately (referring to the bottom in-place inclinometer). But the cost of the measurement is high. In Hong Kong, a few trials were done on automatic monitoring of slope movements using in-place inclinometers. However this technique has not been widely used due to the high cost.

Obviously, displacement is one of the most important parameters for evaluating whether or not a slope is safe. This paper introduces an integrated slope monitoring system which combines (a) an automatic conventional slope monitoring package and (b) a multi-antenna Global Positioning System (GPS) package. The system was installed in a loose fill slope in the New Territories, Hong Kong. The main monitoring results were obtained. The results are presented, discussed and interpreted with results from FLAC numerical modelling in the following sections.

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AN INTEGRATED SLOPE MONITORING SYSTEM

The integrated system consists of (a) an automatic conventional slope monitoring package and (b) a multi-antenna Global Positioning System (GPS) package (Ding et al. 2002). The two packages and their integration are presented as follows.

Automatic Conventional Slope Monitoring Package

The automatic conventional slope monitoring system consists of (a) in-place inclinometers, (b) piezometers, (c) a rain gauge and (d) a Time Domain Reflectometer (TDR). The package has a datalogger, a M20T modem, a power supply and software. Details of the instruments are presented in the following sections.

In-place Inclinometer: In-place inclinometers (model 906 ‘little dipper’, biaxial and high-gain version) were adopted in this project. In-place inclinometers are installed in boreholes, generally at depths less than 45m for purpose of monitoring ground movement. The inclinometers can be installed in a standard inclinometer access tube with an internal diameter of 69.85mm. They can detect the rotation angles at different depths, which are used to obtain slope lateral movements and direction. After the inclinometers have been installed at a slope site, readings can be taken automatically by a data acquisition/control system.

Piezometer: Vibrating wire (VW) piezometers were selected in the project. A VW piezometer can allow remote measurements of piezometric level and pore pressure over a long period of time.

Rainfall gauge: A rain gauge of type TE525 MM was selected to monitor rainfall intensity at a slope site.

TDR 100 set: A Time Domain Reflectometer (TDR) is a new technology, which is very effective and accurate in locating a slip surface in the slope mass. Model TDR 100 was selected as a part of the automated integrated slope monitoring sub-system.

Slope Sentry datalogger: A Model 798-B-TDR Slope Sentry is a stand-alone datalogger for monitoring up to 10 inclinometers, up to 16 vibrating wire piezometers, a tipping bucket rain gauge and up to 8 TDR cables. Its main hardware consists of a CR10X datalogger, an M20 terminal modem, an AVW4 vibrating wire interface and a PS12 power supply. The Slope Sentry provides electric power to all instruments, manages measurement schedule, records the output readings from all instruments and transfers data from the monitoring site to a computer in the office. A CR10 X datalogger performs data recording, sensing of alarm limits and control of an alarm system. An M20 terminal modem is used to transmit data remotely. This modem allows transfer of data across a GSM900 digital cellular phone network. The M20T is a GSM digital transceiver, which is considered to be a GSM mobile telephone without a handset. The M20T has an extra interface to allow a data device to be connected and can be used to collect data from a datalogger at a speed of a land-based 9600 baud rate modem, which closely emulates a standard “Hayes style” telephone modem. Through the M20T modem and the GSM communication technology, temporarily-stored data in the datalogger can be transferred to a computer in the office. Main specifications of the M20 Terminal modern are: standard RS 232 serial port; operating temperature ranges from -29°C to +55°C.

A Multi-antenna GPS Package

A GPS receiver is very expensive. In the conventional approach, one GPS receiver is connected to one antenna for monitoring the movement of one point only. If there are ten points to monitor, ten GPS receivers are needed. This makes GPS monitoring more expensive than conventional geotechnical instruments. The one-receiver-to-one-point approach adopted in the standard GPS is the main reason of the high cost using GPS for monitoring. In our newly developed GPS technology (Ding et al. 2002), a multi-antenna-switch is added to one GPS receiver to build up a multi-antenna GPS array, in which there are eight input ports (antenna) and one output port. After the receiver has got signal from one port, the receiver automatically switches to the next port until data from all ports are received. The one receiver in the multi-antenna GPS works in the same way as that of a standard GPS. Since only one receiver is used instead of 10 receivers, the multi-antenna GPS is much cheaper than the standard GPS with a resulting cost saving of about 80%.

The multi-antenna GPS developed by Ding et al (2002) makes local correction by using a reference GPS receiver with one antenna on stable ground. Both the multi-antenna GPS and the reference GPS have stand alone data storage and communication devices and software. All these form a multi-antenna GPS package.

Integration of the Conventional Package and the Multi-antenna GPS Package

The two packages can work separately and can also be integrated together as one system. The integrated system combines and enhances the advantages of the conventional sub-system and the multi-antenna GPS. Data of the subsurface movements, pore water pressures and the slip location from the conventional sub-system plus rainfall measurements are combined with the data of the three-dimensional (3-D) movements of a large number of points on the slope surface to give a complete set of data, a whole picture, and better understanding of the slope movement. Figure 1 illustrates the integration of the two packages.

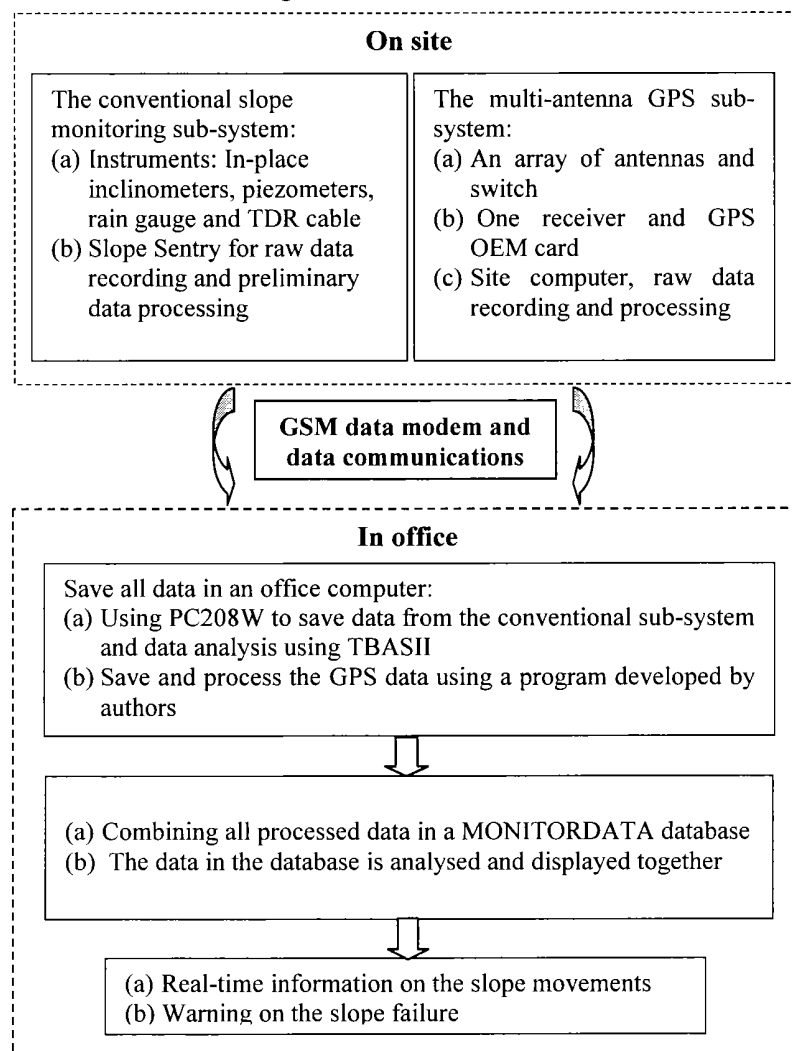


Figure 1. Integration of the two packages

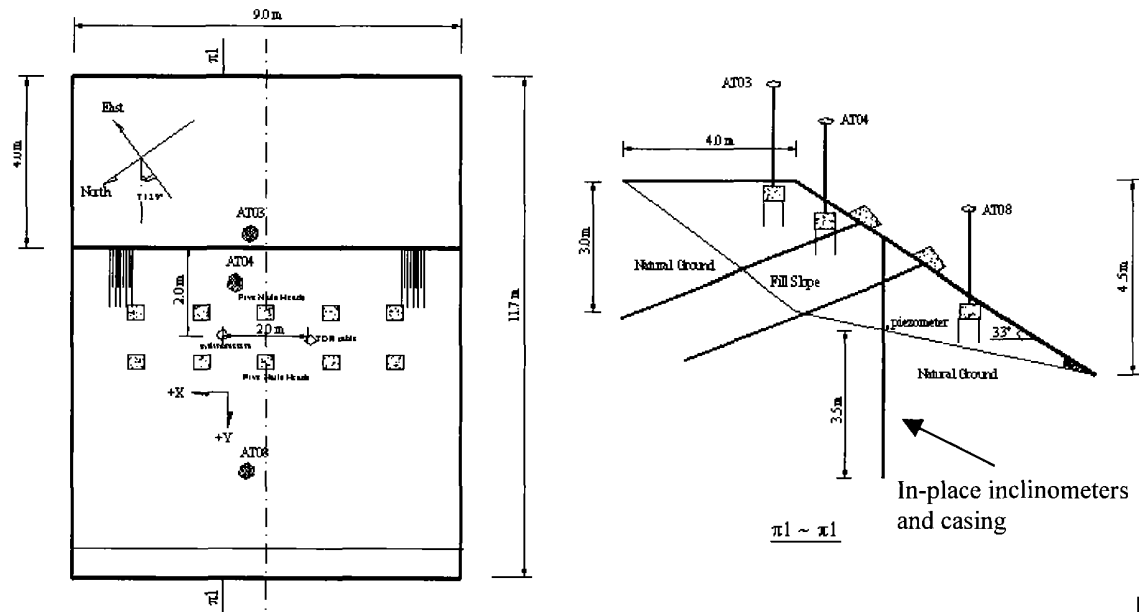


Figure 2. Dimensions of a soil nailed loose fill slope and location of instruments

INSTALLATION OF THE SYSTEM FOR MONITORING A SOIL NAILED LOOSE FILL SLOPE

The above integrated slope monitoring system was installed at a soil nailed loose fill slope test site in the Kadoorie Agricultural Research Centre of the University of Hong Kong (HKU) in Yuen Long, Hong Kong (see Figures 2 and 3). The loose fill slope was part of a research project of the Department of Civil Engineering of HKU (2002). Completely Decomposed Granite (CDG) was loosely placed to form the test slope. The natural ground below the test slope had a slope angle of 15° . The CDG loose fill slope has a slope angle of 33° . In plan view, the slope area was 9.0m (width) $\times$ 11.7m (length) with a maximum height of 4.5m. Two rows of soil nails were installed. There was a platform of 3m $\times$ 9.0m at the slope crest. Precast concrete blocks were used as surcharge acting on the crest platform. The concrete blocks were placed gradually. Ground water table was also raised by water inflow from small pipes directly buried under the slope crest and spreading water on the slope surface as artificial rain using sprinklers. All these were to cause the fill slope to fail under a controlled condition.

As shown in Figures 2, at the slope site, five in-place inclinometers were installed in one vertical borehole of 6.3m in depth into the natural ground beneath the test slope. One TDR cable was embedded in another near-by vertical borehole of 6.5m in depth. One piezometer was directly buried at the interface between the CDG fill and the natural ground next to the TDR and inclinometer boreholes. An array of three steel poles (AT03, AT04, AT08) were installed on the sloping surface. The top of each pole supported one GPS antenna. One additional pole was installed on a stable ground 20m from the slope for a reference GPS antenna. All the Slope Sentry, TDR tester, GPS receivers, switch box and other instruments



Figure 3. Photo of the slope and instruments

were put in a small shed close to the monitoring site. For more information of the monitoring, please see Yin et al. (2002, 2003a,b,c).

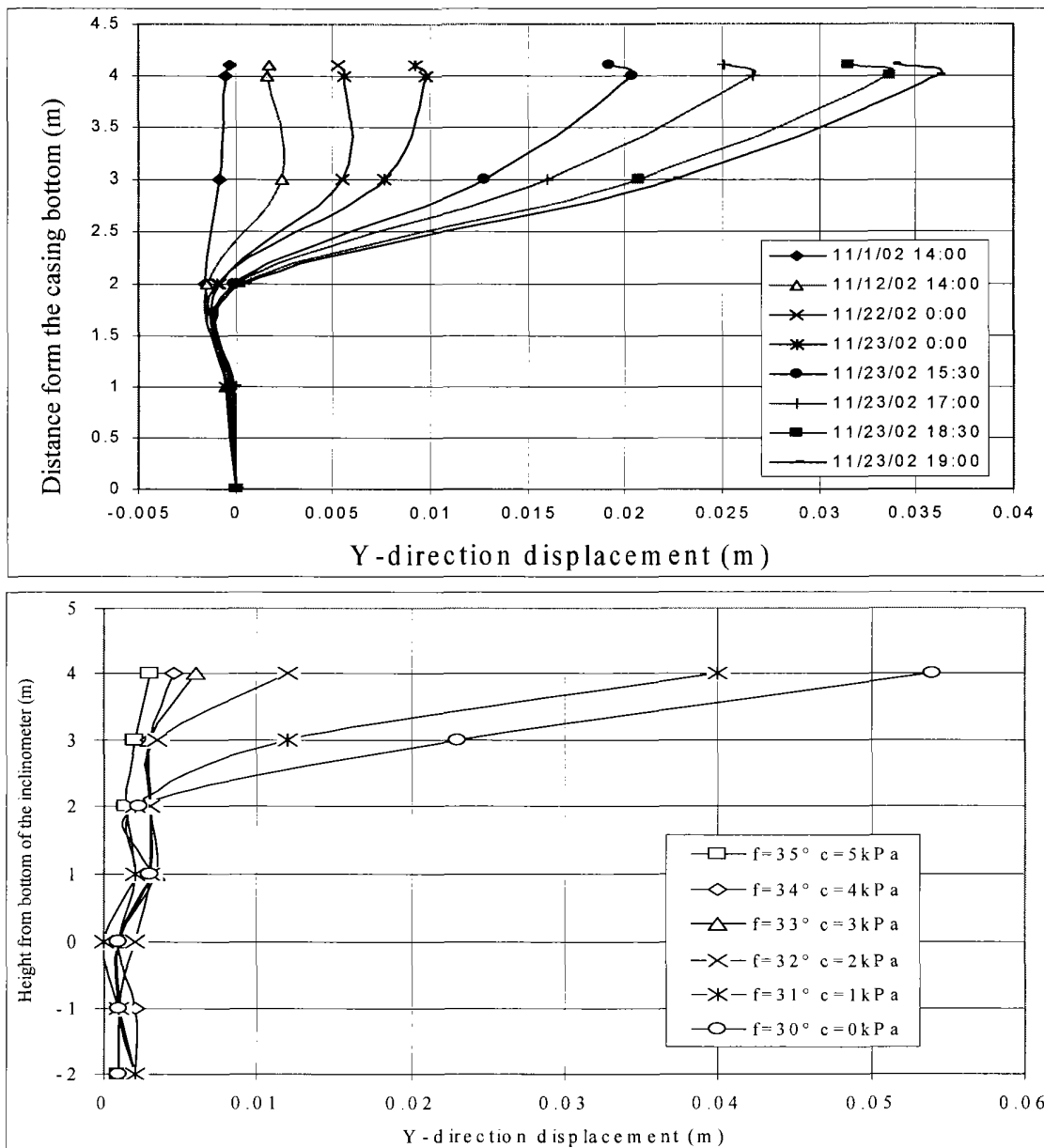


Figure 4. Down-slope movements - (a) from in-place inclinometers (top) and (b) from FLAC modeling (bottom)

MAIN MONITORING RESULTS

The loose fill slope testing (spreading water, placing concrete blocks on the slope crest, etc.) started August 2002 and ended at 7:00pm, local time, on 23 November 2002, for almost four months. Both the conventional instruments and the GPS measurement were checked before monitoring. Main results are presented in Figures 4, 5 and 7.

The lateral displacement profiles with time are obtained from the five in-place inclinometers in on vertical casing as shown in Figures 2 and 3. The profile of down-slope lateral movements measured along the in-place inclinometers is plotted in Figure 4 (top) at 8 different times. Figure 5 presents the curves of Y & X direction displacements with time at

the top point (on the slope surface) of the in-place inclinometer casing. It is noted that the positive Y-direction (+Y) is the down slope direction. The X-direction is perpendicular to the Y-direction. From Figure 5, the +Y-direction horizontal displacement increases with time. The rate of displacement is accelerated at displacement of 9mm. The maximum +Y-displacement is 34mm at failure.

Figure 6 shows the curve of Y-direction displacement with time at GPS point AT03 (on the slope crest). From Figure 6, the +Y-direction horizontal displacement increases with time. The rate of displacement is accelerated at displacement of 50mm. The maximum +Y-displacement is 180 mm at failure. Both the in-place inclinometers and the GPS AT03 have indicated that the horizontal displacement has an acceleration rate before failure. This value may be considered to be a critical value for landslide warning. It is noted that the critical displacement and the maximum displacement at failure may be different at different points.

HKU installed independent in-place inclinometers at the slope crest (close to AT03) and at the middle of the slope (close to our in-place inclinometer casing). It is found that results from HKU are similar to the results in Figures 5 and 6. This supports the reliability and accuracy of our integrated system from another independent system.

FLAC NUMERICAL MODELLING OF SLOPE MOVEMENTS AND COMPARISON WITH MONITORING RESULTS

Figure 7 (a) shows (a) the soil profile and dimensions of the soil nailed loose fill slope and (b) FLAC grid. The soil nails and surcharge using blocks are not shown, but can be

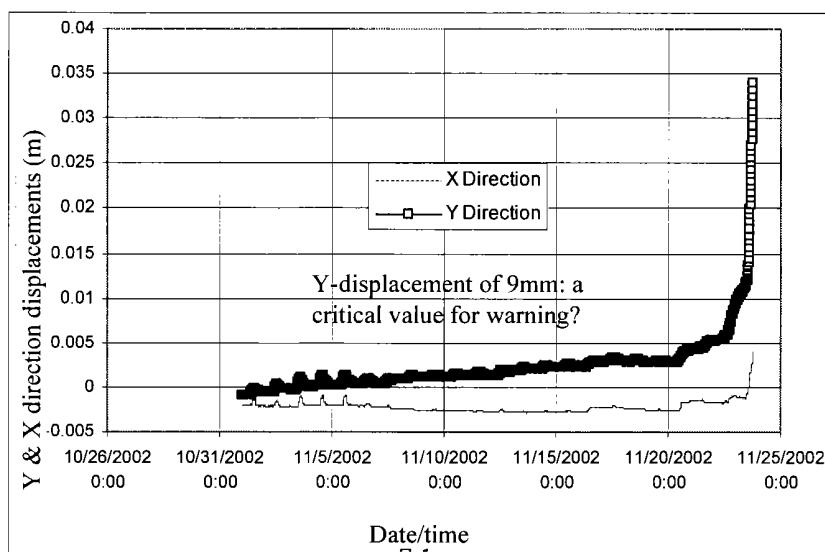


Figure 5. Y & X direction displacements - time at the in-place inclinometer casing top point on the slope surface (+Y: down slope) (in-place inclinometer measurement)

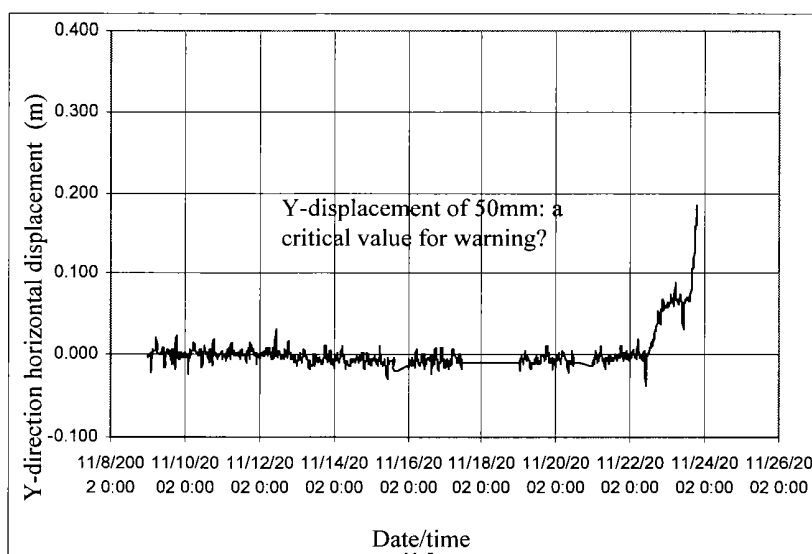


Figure 6. AT03 Y-direction horizontal displacement (+Y: down slope) (GPS measurement)

found in Figures 2 and 3. The model parameters for the CDG loose fill and the natural soil are summarized in Table 1. The model parameters for the soil nails are listed in Table 2.

Figure 7 illustrates the displacement profiles along the in-place inclinometer casing for six sets of friction angles (35° to 30°) and cohesions (5 kPa to 0) under full surcharge of 72kPa (5 layers of concrete blocks). The decrease of the friction angle and cohesion is a simulation of the decrease in shear strength as soil suction decreases after gradual artificial rain infiltration into the soil. By comparing the FLAC results with monitored results, it is found that when friction angle is 31° and cohesion is 1kPa, the lateral down slope movement is close to the monitored value.

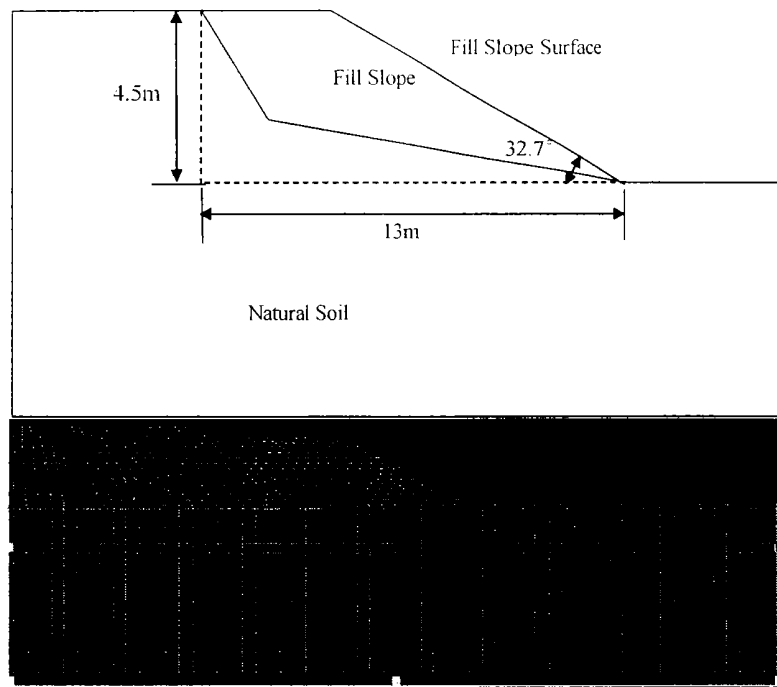


Figure 7. (a) Soil profile and dimensions of the slope (top) and (b) FLAC (bottom)

REMARKS

An integrated slope monitoring system has been briefly introduced. Main results from field monitoring and FLAC modelling are presented and discussed. From the work presented, the authors may make following remarks:

- The integrated system takes advantages of the conventional slope monitoring instruments and the GPS measurements and avoids the short-comings of the two packages. The integrated system is more cost-effective for slope monitoring.
- The integration approach is flexible and practical.

Table 1. Parameters for soils

	Fill	Natural soil
Young's modulus, E (kPa)	30,000	36,000
Poisson's ratio, ν	0.3	0.3
Bulk Modulus, K (kPa)	25,000	30,000
Shear Modulus, G (kPa)	11,500	14,000
Bulk density ρ (Mg/m^3)	1.8	1.8
Friction angle ϕ	$30^\circ \sim 35^\circ$	$30^\circ \sim 35^\circ$
Cohesion, c (kPa)	0 to 5	0 to 5
Tension (kPa)	0	0

Table 2. Parameters for soil nails

Young's modulus of steel, E (GPa)	200
Yong's modulus of concrete, E , (GPa)	30
Overall Yong's modulus of soil nail, E (GPa)	40
Ultimate yield stress of steel (MPa)	425
Shear bond stress (GPa)	10
Shear friction angle ϕ	$30^\circ \sim 35^\circ$
Soil nail rebar diameter (mm)	25
Grouted soil nail diameter (mm)	100

- integrated system is reliable and accurate enough for slope monitoring.
- (d) The monitoring results show that the down slope horizontal displacements are accelerated at certain critical values before failure. Both the critical displacements and the maximum displacements are, however, different at different locations of the slope.
 - (e) The FLAC modelling results indicate that the critical friction angle was probably 31° and cohesion was only 1kPa at the slope failure. The reduction in friction angle and cohesion was due to reduction of soil suction after artificial rain infiltration.

REFERENCES

- Ding, X.L., D.F. Huang, J.H. Yin, Y.Q. Chen, C.K. Lau, Y.W. Yang, Y.R. Sun and W. Chen (2002). A new generation of multi-antenna GPS system for landslide and structural deformation monitoring. Proceedings of Advanced Building Technology 2002 (ABT2002), 4-6 Dec 2002, Hong Kong. In "Advances in Building Technology" (edited by M. Anson, J.M. KO, and E.S.S. Lam) by Elsevier, pp.1611-1618.
- Department of Civil Engineering of the University of Hong Kong (2002). Drawings of the "Full-scale Field Tests of Loose Fill Slope Stabilised with Soil Nails at The Kadoorie Agricultural Research Centre, Lam Kam Road, Shek Kong, Yuen Long".
- Malone A. (1998). Slope movement and failure: evidence from field observation of landslides associated with hillsides cuttings in saprolite. in Hong Kong. Proceedings 13th Southeast Asia Geotechnical Conferences, Taipei.
- Yin, J-H., X.L. Ding, YW Yang, CK Lau, DF Huang, and YQ Chen (2002). An integrated system for slope monitoring and warning in Hong Kong. Proceedings of Advanced Building Technology 2002 (ABT2002), 4-6 Dec 2002, Hong Kong. In "Advances in Building Technology" (edited by M. Anson, J.M. KO, and E.S.S. Lam) by Elsevier, pp.1661-1670.
- Yin, J-H., Ding, X.L. and Yang, Y.W. (2003). Application of a global positioning system and conventional instruments for remote monitoring and warning of slope movements. Journal of Hazard Prevention and Mitigation Engineering, Vol.23, No.2, pp14-21.
- Yin, J-H, XL Ding, YW Yang and YQ Chen (2003a). Monitoring a soil nail stabilized loose fill slope using an integrated slope monitoring system. In Proc of "International Conference on Slope Engineering", (Edited by CF Lee and LG Tham), published by Dept of Civil Eng, University of Hong Kong, Vol.2, pp.998-1004.
- Yin, J-H., Ding, X.L., Yang, Y.W., Lau, C.K., Huang, D.F., Chen, Y.Q. (2003b). Integration of conventional instruments and global positioning system for automatic monitoring of slopes. Chinese Journal of Rock Mechanics and Engineering, Vol.23, No.3, pp.357-364.
- Yin, J-H., X.L. Ding, Yuwen Yang, and YQ Chen (2003c). Monitoring a soil-nailed Hong Kong using GPS and conventional instruments (in Chinese). In Proceedings of The 3rd Cross Strait Conference on Structural and Geotechnical Engineering, October 23 25, 2003, Taipei, Taiwan, Edited by YB Yang, LZ Lu, and SY Zhu (ISBN: 957-29116-0-0), pp535--540.

ACKNOWLEDGEMENTS

The research work in this paper is supported by grants from the Research Grants Council of the Hong Kong SAR Government and The Hong Kong Polytechnic University. The authors thank the Department of Civil Engineering and The Kadoorie Agricultural Research Centre of The University of Hong Kong for support on the field trial of the slope monitoring system. The authors would thank Dr. CK Lau for advice on the project, Dr. DF Huang for his early work on the GPS measurements and Dr. YW Yang for field monitoring works.

TESTING STUDY ON THE INTERFACE BETWEEN SOIL AND SOIL NAIL

Lok Man Chu¹ and Jian-Hua Yin²

Abstract: In Hong Kong, soil nailing has become a well-accepted technique for improving sub-standard slopes which are considered liable to failure. One of the most important parameters for the design and safety assessment of a soil nailed slope is the ultimate shear strength at the interface between soil and cement grout. However, in practice, the shear strength is simply assumed to be a function of the effective angle of internal friction of the soil. This interface resistance estimation could be on the risky side or the conservative side, depending on a number of influencing factors, such as, cement surface (*e.g.* dry, wet, waviness), soil conditions (*e.g.* dry, wet, relative density), grouting lag time and stress conditions. An experimental study has been carried out to investigate the ultimate shear resistance between soil and cement grout by direct shear box interface shear tests and pull-out tests. These laboratory tests aimed at studying the influence of relative density, water content, overburden pressure and surface waviness of the nail at the soil and cement grout interface. The results show that the ultimate shear resistance depends on the overburden pressure and the soil moisture contents among other factors. A correlation of the interface shear strength with major influencing factors is then derived for practice design applications.

INTRODUCTION

In the mid 1980's, the technique of soil nailing was introduced to Hong Kong and has become the most popular solution to the conventional slope stabilization comparing with recompaction, using earth retaining structure, or reducing inclination of the slope *etc.* (Yim et al. 1988; Smith 1992, Chu 2003; Chu and Yin 2003). Generally, the design of a soil nailed structure requires appropriate design parameters, specifically soil-to-nail interaction parameters *i.e.*, the interface shear strength parameters (soil adhesion c'_a and angle of interface friction δ'). However, there is a lack of quantitative data and understanding of the interface shear strength of soil nails.

For the current design of a soil nail system, the pull-out capacity of a soil nail is estimated from an assumed shear strength, and then verified by field pull-out tests during the construction stage (Powell and Watkins 1990). The interface shear strength parameters of the soil nail adopted in the design is normally equal to the critical-stage shear strength (apparent cohesion c' and angle of internal friction ϕ') of the soil. This design approach may underestimate or overestimate the pull-out capacity of the soil nail due to the uncertainties in selection and verification of the interface shear strength parameters (soil adhesion c'_a and

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angle of interface friction δ') of the soil nail. To enable an optimization design of a soil nail system, an understanding of the fundamental interaction mechanism between the cement-grouted nail and surrounding soil is necessary.

To provide some insight into the interface behavior between the cement-grouted nail and soil, two types of laboratory tests have been adopted, *i.e.*, pull-out tests and direct shear box interface shear tests. Pull-out tests were carried out using an apparatus specifically designed for laboratory pull-out tests, and interface shear tests were carried out in a large-size direct shear test apparatus. The soil used was a completely decomposed granite (CDG) taken from a soil nailed slope in Hong Kong. The soil-cement grouted shear samples and their preparation were a simulation of installation procedures of a cement-grouted nail in a soil slope. From the test results, shear stress-displacement behaviors were investigated to better understand the interaction mechanism between the cement-grouted nail and CDG soil. Furthermore, the interface shear strength from the interface shear tests were evaluated, and compared with those from the laboratory pull-out tests.

TESTING APPARATUS AND PROCEDURES

Large-size direct shear test box

Soil-soil direct shear tests and soil-cement grout interface shear tests were performed using a large-size direct shear test apparatus. A photograph of the large-size direct shear test apparatus is shown in Fig. 1. A schematic diagram of a soil-cement grouted specimen placed inside the direct shear box is shown in Fig. 2. The direct shear box has a 305mm x 305mm size upper box and a 406mm x 305mm size lower box in plan view. The average shear stress is measured by a 45 ± 0.05 kN load cell and the shear displacement is measured by a 120mm long linear variable differential transducer (LVDT) pointed to a steel rod in the lower halve of the direct shear box. A flexible rubber diaphragm is filled with de-aired water to provide a uniformly distributed vertical pressure on the stiff plastic plate, and the water flowing in or out of the rubber diaphragm is measured as the volume change of the soil sample. Data from all transducers are taken in a digital readout connected to a computer.

To simulate the procedures of a grouted nail installation in a soil slope, cement mortar was poured on a waviness surface of a recompacted CDG soil sample. The soil was compacted to achieve a controlled dry density 1.615Mg/m^3 , which was 95% of the maximum dry density 1.70Mg/m^3 obtained from a standard compaction test. Four different degrees of interface waviness surface were created to simulate the surface waviness of the drillhole and to investigate the effects of surface waviness on the interface shear strength. The angles of interface surface waviness are 0° , 10° , 20° and 30° in a zigzag shape and the width of the wavy “teeth” is 12.5mm. The soil-cement grouted sample was cured for 28 days to achieve matured cement strength, and the cement grout had a good natural bonding with the recompacted soil. For the interface shear test, the soil-cement grouted sample was placed in the direct shear box with the cement grout in the lower box. All tests in the large-size direct

shear test box were conducted at a shear displacement rate of 0.3mm/min.

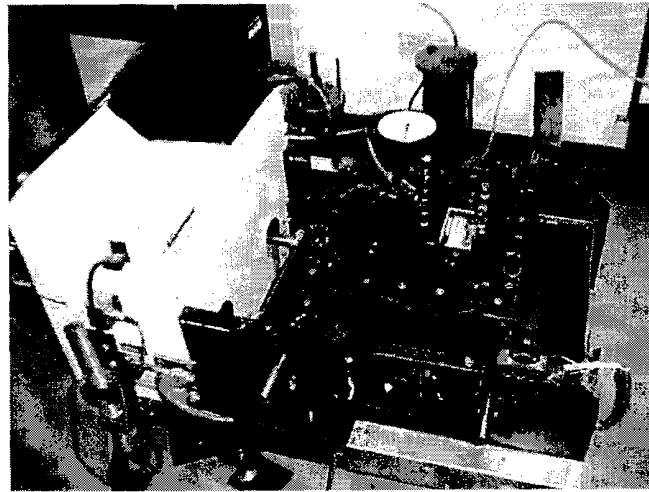


Figure 1. View of a large-size direct shear box test apparatus

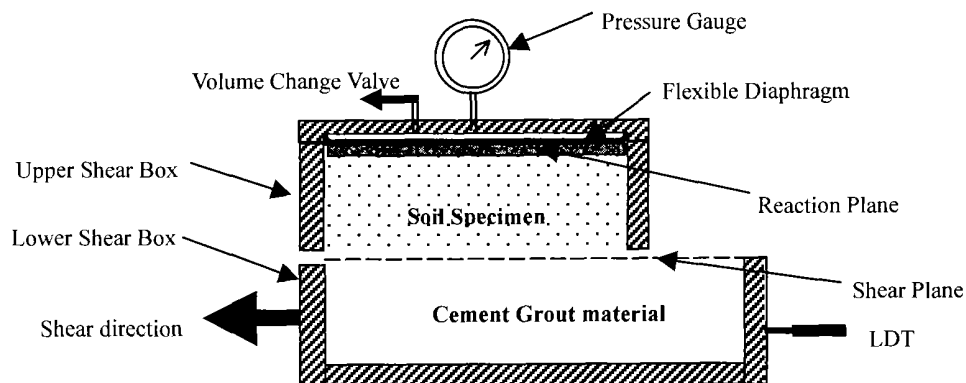


Figure 2. Arrangement of a soil-cement grouted specimen inside the direct shear box

Laboratory pull-out test box

Photo of the laboratory pull-out test apparatus is shown in Fig. 3. Cross-sectional view of the laboratory pull-out test apparatus is shown in Fig. 4. The pull-out test box has internal dimensions of 690mm long by 560mm wide by 605mm high. A rubber diaphragm filled with water was used to produce a uniformly distributed pressure on the compacted soil. The pull-out load was measured by a 100 ± 0.01 kN load cell and a mean pull-out displacement was monitored using linear variable differential transducers (LVDTs) on the nail head. These instruments were linked to data loggers that were used to scan the measurements at desired time intervals.

The soil was first mixed to nearly to the optimum moisture content of 19%, and then placed in the box with six layers and compacted to achieve a relative density of 95%. A 100mm diameter hole was pre-drilled into the compacted soil by a concrete coring machine. A 32mm diameter ribbed steel bar was centralized on the drillhole a cement mortar with a

water-cement ratio of 0.45 was continuously grouted over the drillhole. The grouted nail was cured inside the soil with 28 days. After that, the desired normal pressure was applied and the pull-out test was conducted at a displacement rate of 0.5mm/min.

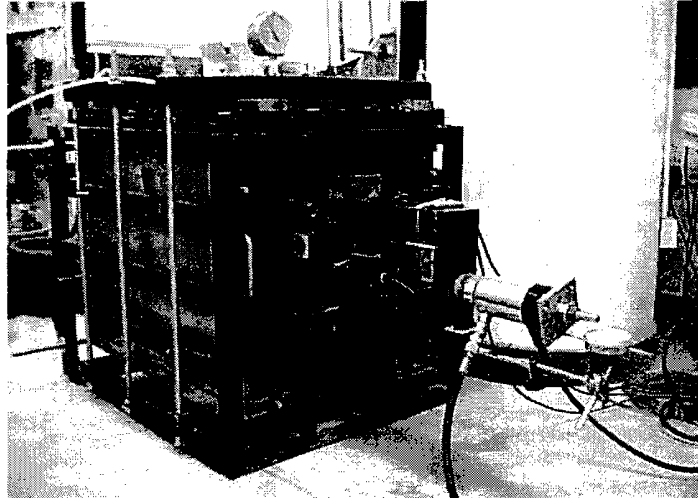


Figure 3. View of a laboratory pull-out test apparatus

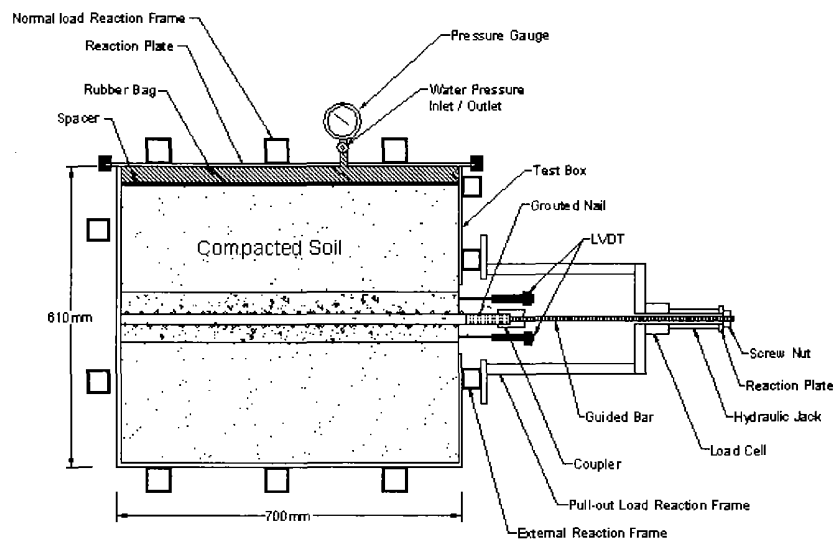


Figure 4. Cross-sectional view of a laboratory pull-out test in the testing apparatus

PROPERTIES OF THE SOIL AND CEMENT GROUT SPECIMENS

The soil used in the laboratory pull-out tests was a Completely Decomposed Granite (CDG), which was excavated from a cut-slope in Highland Park at Kwai Chung, Hong Kong. According to GEO-Guide 3 (GEO 1988), the soil can be classified as yellowish brown, well-graded, clayey gravelly sandy SILT. The shear strength parameters (c' , ϕ') of the CDG material are presented in Table 2. Cement grout used for the tests was prepared from a cement mortar with a water-cement ratio w/c of 0.45. The properties of the CDG and cement grouted material are summarized in Table 1.

Table 1. Material properties of the CDG and cement grouted material

Completely decomposed granite (CDG)		
Specific gravity (G_s)		2.64
Maximum dry density ($\rho_{d(max.)}$)	Mg/m ³	1.70
Optimum moisture content ($w_{(opt.)}$)	%	19.00
Optimum air void content ($A_{v(opt.)}$)	%	3.00
Gravel	%	28.22
Sand	%	37.19
Silt	%	19.59
Clay	%	15.00
Plastic limit (w_p)	%	37.08
Liquid limit (w_l)	%	62.25
Permeability (k_{20})	m/s	4.54×10^{-5}
Tangent Young's modulus (E_{50})	MPa	14.00
Poisson's ratio (ν)		0.40
Cement-grouted material		
Density (ρ_d)	Mg/m ³	2.00
Uniaxial compressive strength (σ_c)	MPa	56.81
Tangent Young's modulus (E_{50})	GPa	16.37
Poisson's ratio (ν)		0.28

Table 2. Values of shear strength parameters from the soil-soil direct shear tests, laboratory pull-out tests and soil-cement grout interface shear tests

	Peak shear strength		Shear strength at displacement of 70mm	
	c' or c_a' (kPa)	ϕ' or δ' (degree)	c' or c_a' (kPa)	ϕ' or δ' (degree)
Direct shear tests	45.77	30.43	38.24	30.17
Laboratory pull-out tests (regular surface nail)	63.46	29.48	32.36	26.20
Laboratory pull-out tests (irregular surface nail)	58.21	32.21	36.82	25.27
Interface shear tests (waviness angle 0°)	30.53	28.89	22.21	28.20
Interface shear tests (waviness angle 10°)	54.11	32.06	42.82	31.08
Interface shear tests (waviness angle 20°)	42.98	32.23	41.59	32.17
Interface shear tests (waviness angle 30°)	43.09	32.68	36.32	32.87

PULL-OUT CAPACITY OF A SOIL NAIL

Chu (2003) have studied the normal stress acting around a 100mm diameter of soil nail inside the laboratory pull-out tests box by a two-dimensional explicit finite difference program – FLAC (1996). The result showed that the value of the normal stress acting on the soil nail σ'_n is equivalent to the overburden pressure σ'_v , which is the stress of the self-weight of the soil plus the applied stress on the soil sample.

The average pull-out shear stress τ_s (kPa) of a soil nail can be calculated from a measured pull-out load F (kN) divided by the active nail surface area A (m²):

$$\tau_s = \frac{F}{A} = \frac{F}{(\pi D)L_s} \quad (1)$$

where L_s is the embedded length of the nail, and D is the diameter of the nail.

In design practice, the following equation is used for calculation of a soil nail pull-out capacity per line meter T (kN/m) (Wong 1995):

$$T = (\pi D c' + 2 D \sigma'_v \tan \phi') \quad (2)$$

where c' is the soil cohesion and ϕ' is the internal angle of the soil friction.

In this paper, the following equation is proposed for the evaluation of the pull-out capacity per meter T (kN/m) of a soil nail, considering the interface shear strength between the nail and soil:

$$T = (\pi D c'_a + 2 D \sigma'_v \tan \delta'') \quad (3)$$

where c'_a is the soil adhesion at the interface, and δ'' is the angle of the interface friction.

From Eqs. (1) and (3), the average shear strength is

$$\tau_s = \frac{F}{A} = \frac{T}{\pi D} = c'_a + \sigma'_v \left(\frac{2}{\pi} \tan \delta'' \right) = c'_a + \sigma'_v \tan \delta' \quad (4)$$

where $\tan \delta' = (2/\pi) \tan \delta''$ (5)

Eq. (4) can be used to fit the shear strength data points to find the parameters of c'_a and δ' . Use the relationship of $\tan \delta'' = (\pi/2) \tan \delta'$, δ'' can be evaluated.

TEST RESULTS AND DISCUSSION

Shear Stress-displacement Behaviors

Fig. 5 shows a comparison of results from direct shear tests (pure CDG soil), interface shear tests (horizontal interface plane with zero waviness angle) and laboratory pull-out tests (regular surface nail) over a range of normal stresses. The average moisture contents of the soil for these tests were controlled nearly to the optimum soil moisture content of 19%.

The shear stress–displacement curves in the soil-cement grout interface shear test results showed similar trend and shape to those from the soil-soil direct shear test results, but are different from the behaviors in the soil nail pull-out tests. The shear stress-displacement

curves for soil nail pull-out test results exhibit an obvious peak and post-peak shear stress-displacement behavior, whereas the curves for direct shear tests and interface shear tests exhibit a relatively small peak and high post-peak shear strength. This behavior may be due to different confinements in the direct shear testing and in the pull-out testing, and direct shear test box would have a higher restriction on soil particles compared to the pull-out test box. The interface surface area of the soil nail is only $(0.1 \times 3.14) \times 0.5 = 0.17 \text{ m}^2$ for a 0.5 m length of nail or $(0.1 \times 3.14) \times 0.7 = 0.22 \text{ m}^2$ for a 0.7 m length of nail. The plan-view area of the pull-out test box is $0.56 \times 0.69 = 0.386 \text{ m}^2$, while the ratio of the soil nail surface over the pull-out test box plan-view surface is $0.17/0.386 = 44\%$ for a 0.5 m length of nail or $0.22/0.386 = 57\%$ for a 0.7 m length of nail. But the ratio for the direct shear test box is 100%.

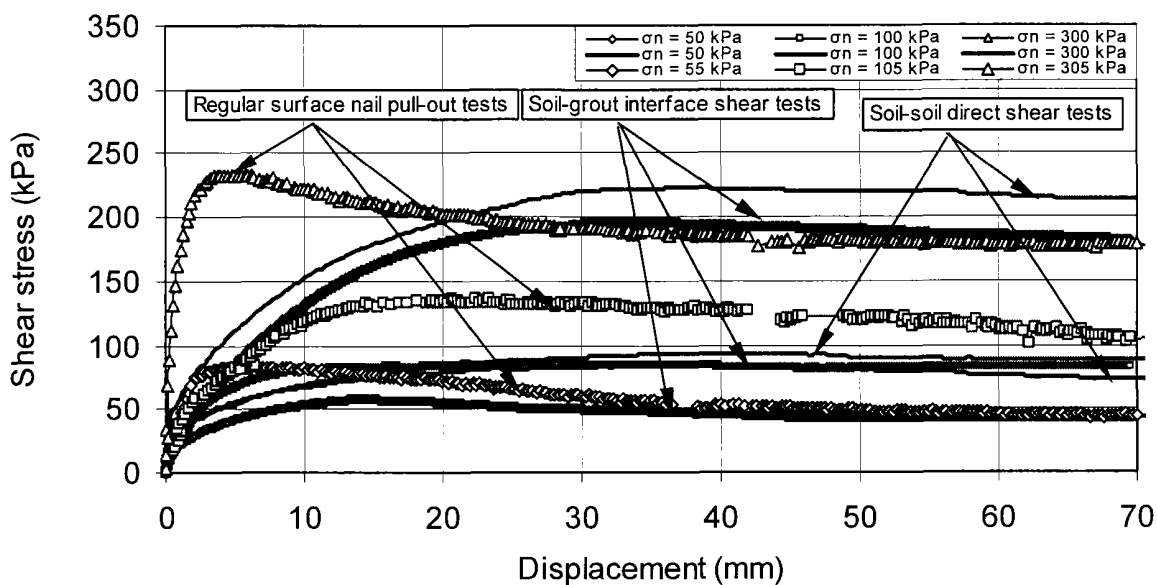


Figure 5. Comparison of shear stress vs. horizontal displacement curves among the soil-soil direct shear tests, soil-cement grout interface shear tests (zero waviness angle) and laboratory pull-out tests (regular surface nail)

The volume changes of the shear samples in the soil-soil direct shear tests, soil-cement grout interface shear tests with zero roughness angle and regular surface nail pull-out tests are shown in Fig. 6. A slightly smaller dilation of shear samples is observed from the pull-out test in a smaller applied normal stress of 50kPa. And a smaller vertical displacement is observed in the pull-out test results compared with the results from the interface shear test and the direct shear test. The maximum vertical displacement in the pull-out tests is about 0.4mm downward under the applied normal stress of 300kPa, but the maximum vertical displacement in the direct shear tests and interface shear tests is about 6mm downward, with the same applied normal stress. The smaller vertical displacement in the pull-out tests may be due to the smaller ratio of the interface shear surface area over the plan-view area of the testing box.

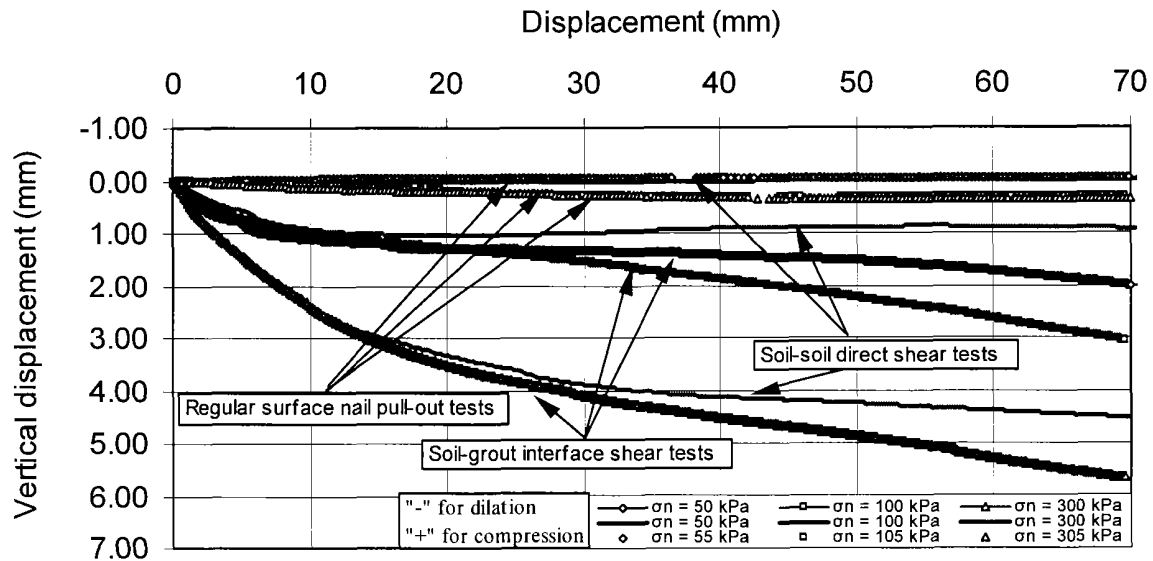


Figure 6. Comparison of vertical displacement vs. horizontal displacement curves among the soil-soil direct shear tests, soil-cement grout interface shear tests (zero waviness angle) and laboratory pull-out tests (regular surface nail)

The “peak shear strength” parameters and the “shear strength at the shear displacement of 70mm” parameters for the pull-out tests, the interface shear tests together with the results from the direct shear tests are summarized in Table 2. It can be seen that the peak interface friction angle obtained from the pull-out tests is generally close to that of the interface shear tests, and both of them are generally close to the peak internal friction angle obtained from the soil-soil direct shear tests.

Effects of Degree of Saturation on Pull-out Shear Strength

A comparison of the pull-out tests results with the different degrees of saturation, S_r , of 70%, 74%, 78% and 86% under the applied normal stress of 300kPa is shown in Fig. 7. It can be observed that a non-linear behaviour and the pull-out shear strength is decreased as the degree of saturation S_r increases. The pull-out shear strength is shown significantly dependent on the degree of saturation of the soil.

For estimating the pull-out shear strength, $\tau_{s,300}$, the following empirical equations were derived:

$$\tau_{s,300} = 3 \times 10^8 \times S_r^{-3.3203} \quad (7)$$

for “peak pull-out shear strength”, and

$$\tau_{s,300} = 1 \times 10^{15} \times S_r^{-7.0788} \quad (8)$$

for “pull-out shear strength at displacement of 200mm”.

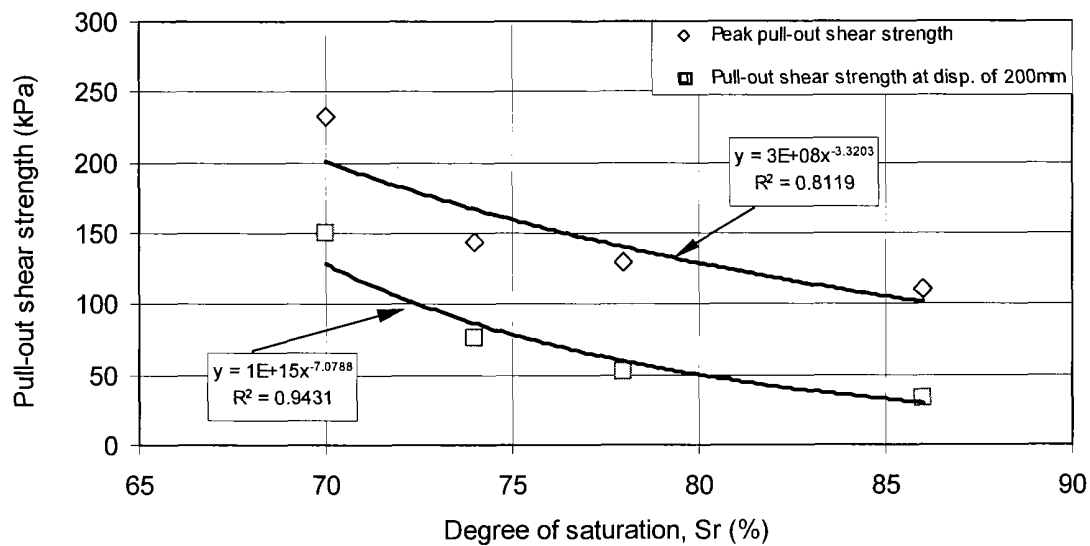


Figure 7. Correlation between pull-out shear strength and the degree of saturation, S_r of 70%, 74%, 78% and 86% under the normal stress of 300kPa

Effects of Surface Waviness on Interface Shear Strength Parameters

Surface waviness of shearing materials is one of the important factors influencing the interface shear behavior (Potyondy 1961; Kulhawy and Peterson 1979; and Uesugi et al. 1990). Fig. 8 shows the effect of cement grouted surface waviness on the interface shear strength parameters from laboratory pull-out tests (regular and irregular surface nails) and interface shear tests (four interface waviness angles 0° , 10° , 20° , and 30°). The results from soil-soil direct shear tests are also presented in the figure for direct comparison. It can be seen that the interface shear strength is influenced by the surface waviness of the cement grout. The shear failure envelope for irregular surface of soil nails is mostly above the shear failure envelope for regular surface of soil nails, and the slope of the failure envelope is increased as the soil-cement grout interface surface waviness increases. The peak interface shear strength can be higher than the shear strength of the soil alone for an irregular surface nails or a cement-grouted interface shear surface with a waviness angle larger than 0° . The results also indicated that there is a great advantage of the surface waviness in the interface friction for a higher interface waviness angle.

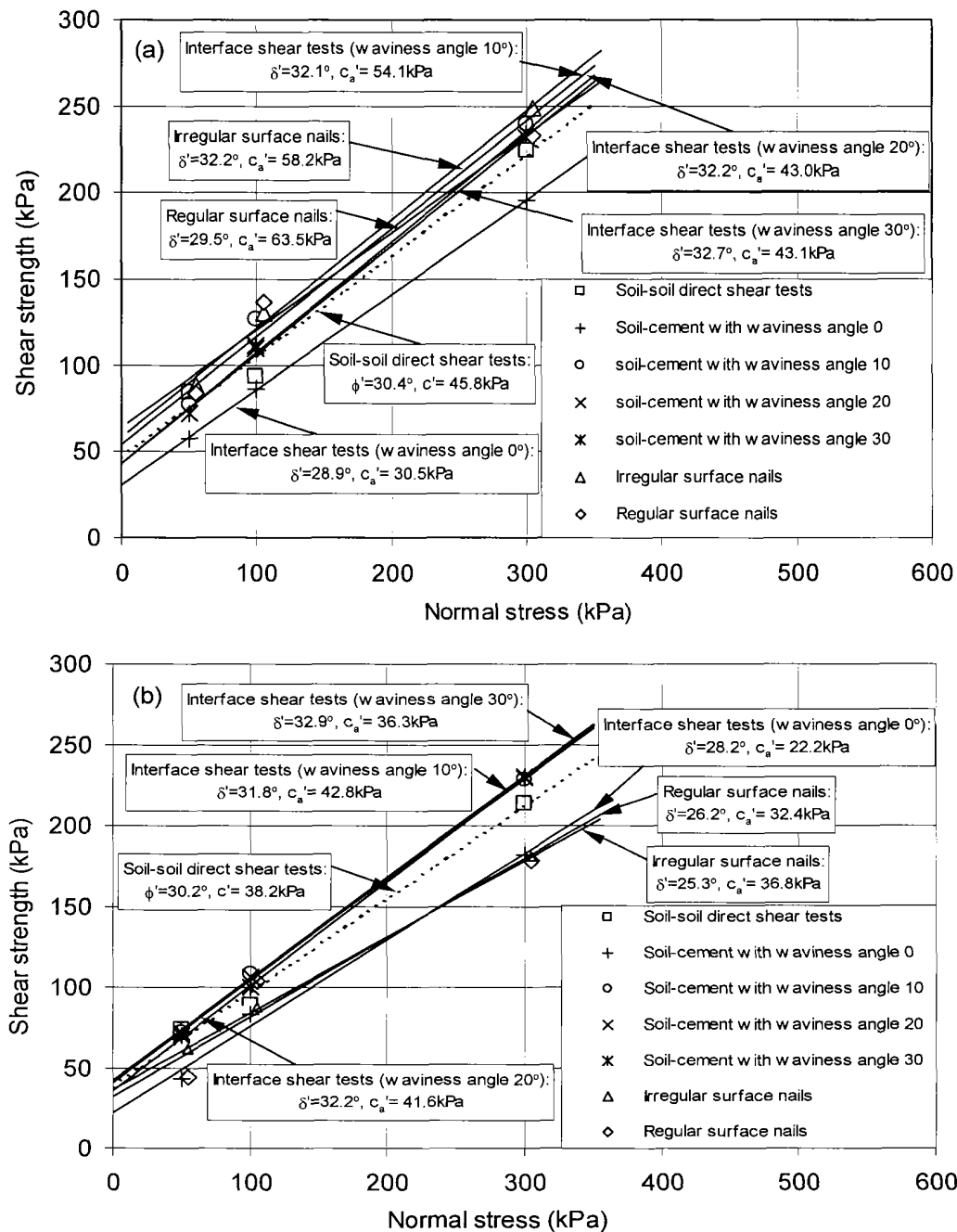


Figure 8. Comparison of shear strength parameters among the tests from soil-soil direct shear tests, soil-cement grout interface shear tests for four different degrees of cement-grouted surface waviness (interface waviness angles 0°, 10°, 20°, and 30°) and laboratory pull-out tests (regular and irregular surface nails) – (a) peak shear strength and (b) shear strength at shear displacement of 70mm

SUMMARY

A modified strength equation based on the Mohr-Coulomb failure criterion is proposed to take into account of the interface shear strength of cement grouted soil nails and it can be used for design of soil nail stabilized slopes. A series of laboratory tests were carried out to

evaluate the interface shear strength between a CDG soil and a cement grout material (soil nail) by using a large-size direct shear test apparatus and a laboratory pull-out test apparatus. Comparisons have been made between interface shear test results and laboratory pull-out test results with respect to shear stress-displacement curves and interface shear strength in relation to the interface surface waviness. The interface shear strength between the nail and soil is shown to be dependent on the overburden stress, the soil degree of saturation, and surface waviness of the cement-grouted material. Empirical equations are derived for the relationship between the pull-out shear strength and the soil degree of saturation (S_r , ranging from 70% to 86%) under the applied normal stress of 300kPa.

REFERENCES

- Chu, L.M. (2003). "Study on the interface shear strength of soil nailing in completely decomposed granite (CDG) soil." MPhil thesis, Sept 2003, The Hong Kong Polytechnic University.
- Chu LM and JH Yin (2003). Shear strength at the interface between soil and cement grout. In Proc of "International Conference on Slope Engineering", (Edited by CF Lee and LG Tham), published by Dept of Civil Eng, University of Hong Kong, Vol.1, pp.338-343.
- FLAC (1996). "Fast Lagrangian Analysis of Continua." User Manual, *Itasca Consulting Group, Inc.*, Minneapolis, Minnesota, U.S.A.
- GEO (1988). "Guide to Rock and Soil Descriptions (Geoguide 3)." *Geotechnical Engineering Office*, Civil Engineering Department, The Government of HKSAR, 186.
- Kulhawy, F.H. and Peterson, M.S. (1979). "Behavior of sand-concrete interfaces." *Proc. 6th Pan-am Conf. on Soil mechanics and Foundation Engineering*, Vol.VII, 225-236.
- Potyondy, J.G. (1961) "Skin friction between various soils and construction materials." *Geotechnique*, Vol.11, No.4, 339-353.
- Powell, G.E. and Watkins, A.T. (1990). "Improvement of marginally stable existing slopes by soil nailing in Hong Kong." *Proc. of the Int. on Reinforced Soil*, Glasgow, 241-247.
- Smith, M.J. (1992) "The effects of variable geology on soil nail pull-out test results in Hong Kong." *Proc. of the Int. Symp. on Earth Reinforcement Practice*, Kyushu, Japan, Vol.1, 543-548.
- Uesugi, M., Kishida, H. and Uchikawa, Y. (1990). "Friction between dry sand and concrete under monotonic and repeated loading." *Soil and Foundations*, Vol.30, No.1, 115-128.
- Wong, H.N. and Pang, L.S. (1996) "Application of prescriptive measures to soil cut slopes." *GEO Report No.56*, Geotechnical Engineering Office, Civil Engineering Department, Hong Kong.
- Yim, K.P., Watkins, A.T. and Powell, G.E. (1988). "Insitu ground reinforcement for slope improvement in Hong Kong." *Int. Geotechnical Symp. on Theory and Practice of Earth Reinforcement*, Fukuoka, Japan, 363-368.

ACKNOWLEDGEMENTS

The work presented in this paper has received financial supports from a RGC grant and from The Hong Kong Polytechnic University.