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Ground Improvement – A Green Technology towards a Sustainable Housing, Infrastructure and Utilities Developments in Malaysia

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ABSTRACT: In the rapid expansion in housing, infrastructure and utilities developments in the last 30 years, engineers have to deal with less favourable sites such as coastal lowlands, swamps, filled ground, reclaimed land, etc. A number of mega size infrastructure projects such as the construction of the 966-km North-South Expressway, the 179-km electrified double-tracking railway project between Rawang and Ipoh, etc. would have been economically non-viable and/or technically non-feasible if they had been constructed using conventional methods meant for good soil conditions. For these mega projects and other similar projects, it was necessary to explore the innovations of using non-conventional methods when poor soil conditions may impair the integrity and serviceability of the structures. In such situations, the natural condition of poor soil needs to be altered to meet the project requirements where settlement requirements are more stringent and poor ground strength needs to be significantly improved. This is termed as ground improvement. The common types of ground improvement used are described in this paper. Due to the increasing awareness of the construction impact on the environment, sustainable construction techniques using green technology such as ground improvement is also increasingly used. A carbon footprint auditing system is introduced for some of the commonly used ground improvement methods.

1. INTRODUCTION

1.1 Malaysian Construction Industry in the Past Decades

When Malaysia attained its Independence in 1957 the economy was fundamentally primary commodity-based with heavy dependence on rubber and tin which contributed about 70% of total export earnings, 28% of government revenue and 36% of total employment (EPU, 2003). The economy remained highly dependent on foreign trade to generate foreign exchange earnings to finance its development.

During the period of 1984-1990, the government instituted major structural adjustments in the economy. Public sector expenditure was restrained to reduce budgetary deficits. Private sector led growth strategy was adopted. This included economic liberalization and deregulation and improving investment policies and incentives to promote private sector participation. Privatization of public sector activities, agencies and enterprises was introduced.

Except in 1998 when the economy was adversely affected by the Asian Financial Crisis, there was generally a sustainable economic growth. The 7th Malaysia Plan (1996-2000) followed by the 8th Malaysia Plan (2001-2005) were implemented during this period to steer the nation's development agenda to achieve the challenges of Vision 2020 which laid out the directions for Malaysia to become a fully developed nation by 2020.

Housing development became a priority in Malaysia's development programs. It aimed at improving the quality of life. Various housing developments were undertaken by both the public and private sectors. While the private sector focused more on overall market demand, the public sector continued to provide houses for sale or rent to the low-income group. During the period of 1996-2005, approximately 1,642,000 new houses were built for the growing population, formation of new households and the replacement of existing old houses. Total expenditure amounted to approximately RM15 billion (US$1 = RM3.80) for housing and other social services (EPU, 2003).

Development of infrastructure and utilities was focused on capacity expansion to meet demand. The higher than expected demand necessitated the adoption of fast track implementation processes, application of new and adapted technologies, reduction of processing time as well as the accelerated privatization of projects. The design and build method was used to fast track the construction of projects while in some mega projects the Built, Operate and Transfer procurement method was used where the financing of projects was facilitated by development financial institutions through privatization and the deferred payment scheme. In the 7th and 8th Malaysia Plans, the government had provided substantial allocation of funds for infrastructure and utilities developments (see Table 1).

Table 1. Development allocation for infrastructure and utilities (EPU, 2003)

	7th Malaysia Plan (1996-2000) Expenditure (RM millions)	8th Malaysia Plan (2001-2005) Estimated Expenditure (RM millions)
Transport:		
Roads	12,270	18,614
Rail	5,450	6,301
Ports	1,089	3,041
Airports	1,271	2.055
Utilities:		
Water Supply	2,383	4,810
Sewerage	665	1,666

The national road network increased from 61,387 km in 1995 to 75,160 km in 2003 with another 1,640 km due for completion in 2005-06. Infrastructural works for railway development included double tracking, strengthening and electrification of tracks. Port development continued to focus on expanding capacity, upgrading and increasing equipment and facilities. The total tonnage of cargo handled increased from 152 million tons in 1995 to 481 million tons in 2005. During this period, about RM500 million was spent on dredging and reclamation works. Airport development was required to expand capacity and upgrade existing facilities. The air passenger traffic grew from 27.3 million in 1995 to 41.6 million in 2005. The air cargo traffic grew from 482,030 tons in 1995 to 1,129,152 tons in 2005. The other utilities development included water supply, sewerage services and communication. The year 2007 will see the completion of the RM1.93 billion Stormwater Management and Road Tunnel (SMART) project.

The Malaysian construction industry has witnessed a strong growth in the back of a higher government expenditure on infrastructure projects and increased construction of residential properties. Employment in the construction sector has recorded an average annual growth of 1.7% contributing 4.4% of employment creation or 38,700 new jobs (EPU, 2003). Hence, there is an urgent need to improve and upgrade our construction technology with the application of new and adapted techniques and ground improvement is one area which has contributed to the nation's development and proves to be the mainstay of providing green technology solutions for sustainability.

1.2 The Role of Ground Improvement

In the early days of development, only the best available lands having reasonably good soil conditions are being developed. This is due to the owners and engineers who have had past experience with the high cost of foundations on poor soils. For example, developers tend to shy away from building on ex-mining lands that demand higher investment cost on foundation. In fact, potential foundation problems have played a significant role in site selection. If a site investigation has shown that the soil conditions have been found unusually bad, it has been prudent to move to a more favourable site.

However, due to the rapid development as described above, the relative importance of good soil conditions in site selection has diminished. The growing scarcity of sites having good soil conditions had made it necessary to utilize all the remaining land regardless of its soil conditions. Some sites are now being developed that were once tin mining lands underlain by soft slime. With the increasingly large scale development of many housing and infrastructure makes it necessary to incorporate both good and bad soil conditions in a single project. Other factor such as the demand for access to deep water has made it necessary to develop ports and container terminals at coastal areas which are very often unfavorable swamps close to water channels. The demands for roads connecting remote towns with cities have forced construction into areas that may not have good soil conditions at all. Therefore, it is becoming apparent that increasing use must be made of sites that previously had been considered unsuitable.

For these unsuitable sites (also referred to as marginal sites), most often, the ground imposes restrictions on the design and the engineer has, apart from abandoning the project, four options: (1) to replace the poor soils with suitable fill materials; (2) to bypass the unsuitable soil by using piles or deep foundations; (3) to redesign the structure to meet the ground limitations; or (4) to alter the natural condition of the poor soil to meet the project requirements. The latter is often termed as ground improvement.

The partial or complete excavation of unsuitable soils and their replacement with better fill materials may be considered. Fine or coarse grained soils can be used as backfilling materials if the ground water level is located below the excavation. Granular materials should be used when the ground water level is high. Complete replacement is generally suitable for sites with shallow deposit of unsuitable soils, usually less than 3m depth. In exceptional cases, it may exceed 3m as in the case of the Kuala Lumpur International Airport as shown in Fig. 1. The depth of excavation is limited to the depth of open excavation without side supports. For deep-seated soft soils deposit, the disadvantage of this method is the need to maintain the stability of the side slopes and to cope with the ground and surface water that accumulate inside the excavation. Besides, the problems faced with disposal of excavated materials especially in urban areas and the increasing cost of imported suitable fill materials (usually sand) may have a bearing on the overall feasibility and economy of this method.

Structural solutions either adopting a deep foundation or a change in the structural design is usually not an economical option. In the case of constructing a road embankment, partial structural solution is for the embankment to rest on piled supported concrete caps or rafts. A full structural solution is the construction of viaducts.

Ground improvement is a viable alternative to conventional structural support solution. In most instances, it proves to be the more economical solution. The main functions of ground improvement are: (1) to control deformation and accelerate consolidation; (2) to increase bearing capacity and to provide lateral stability; and (3) to increase resistance to liquefaction. Liquefaction has become important in view of the increasing incidents of tremors (due to seismicity in neighbouring country) that have been felt in Malaysia in recent years.

The above main functions can be accomplished by modifying the soil's characteristics with or without the addition of imported materials. Improving the soils at the surface is usually an easy task and relatively inexpensive. When at depth, the task becomes more difficult. It usually requires more rigorous analyses and the use of specialized equipment and construction procedure. Local experience is also important.

One of the earliest published applications of ground improvement in Malaysia dates back to 1978 for a housing development project on dynamic consolidation (Ting, 1982 and Ting *et al.*, 1982). This early case history started a rapidly expanding body of practice on ground improvement in the years after.

Figure 1. Excavation and replacement of unsuitable materials

2. MARGINAL SITES FOR IMPROVEMENT

The word "marginal" is a relative term. G.F. Sowers once referred it as "almost any engineering quality of soil that is sufficiently poor that foundation costs are unusually high that special technique of foundation treatment must be utilized or that the risks of future trouble are great might be termed marginal". Furthermore, ground improvement need not be necessarily applied to sites having poor soil conditions. It may happen that a medium ground, which may not require improvement at a given load, may prove to be inadequate in relation to a higher imposed load.

Ting (1998) has presented a comprehensive list of sites that may require improvement:

(i) Filled ground – When a natural stratum is excavated and/or deposited as fill without compaction, the resulting filled-up ground can often be deficient. Non-compacted fill is in a loose state and partially saturated. When saturated by infiltration of water, "collapse" settlement will take place under applied load and the self-weight of the fill;

(ii) Disturbed ground – This mainly refers to natural ground that has been disturbed by mining activities such as in tin mining operations. The problematic soils are the loose sand and soft slime materials that are deposited in tailing ponds after the processing operations. The loose sand tends to deposit nearer to the discharge point of the tailings due to its weight while the soft slime tends to deposit further away.

(iii) Infilled valley – This is usually refers to present-day valleys that contain soft alluvium deposited in the past.

(iv) Riverine deposit – This refers to recent deposits within a general watercourse that has been repeatedly deposited in times of flood and recession of waters. They are usually granular materials mingled with clayey materials.

(v) Coastal and estuarine deposit – They are usually very loose to loose silty sand often presents as coastal deposits and soft marine clay that occurs both as coastal and estuarine deposits. Peats are also encountered in coastal, estuarine as well as inland deposit

3. GROUND IMPROVEMENT METHODS

The Malaysian practice of ground improvement is generally divided into 3 main categories: consolidation, densification (compaction) and reinforcement. Consolidation of soft cohesive soil is achieved through surcharging using fill materials with the use of vertical drains. Vacuum pressure replaces or supplements the surcharge fill materials in a vacuum consolidation process. Densification which applies mainly to loose granular soil includes dynamic compaction and vibro compaction. Soil reinforcement is further divided into 2 groups: (1) non-rigid inclusions are those involving granular backfill materials (i.e. dynamic replacement and vibro replacement columns); and (2) semi-rigid inclusions are those involving cement grout (i.e. deep soil mixing and controlled modulus columns). Fig. 2 shows the common techniques of ground improvement used.

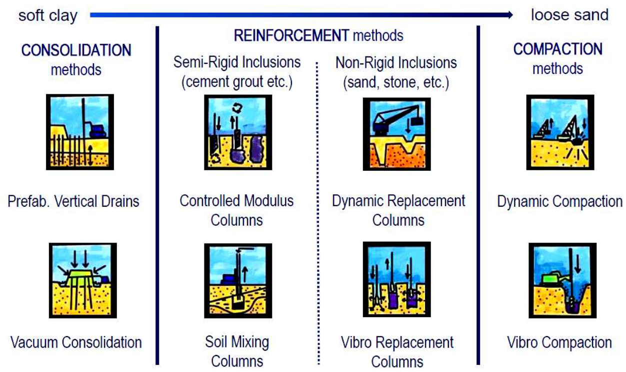

Figure 2. Common ground improvement techniques

Some techniques are more suitable for one type of soil while others apply to a wider range of soil. Compaction methods such as dynamic compaction and vibro compaction aim to improve loose granular soil. Needless to say, they are suitable for densification of loose sand which is susceptible to liquefaction, are generally not used at all for improvement of saturated soft clay, and vice versa. In practice, it may require the selection of one or a combination of techniques to meet the project requirements if the soil conditions vary much on site as each technique has its own merits, limitations and economies.

4. CONSOLIDATION METHODS

Deep deposits of soft cohesive soil are generally located on low lying coastal and deltaic areas as shown in Fig. 3. Some of the recent flood plain deposits along old rivers may also contain localized deposit. The thickness varies from 5m to 25m and in some locations may exceed 30m thick. These deposits are very soft clay either of marine or alluvial origin.

The pertinent characteristic of this soft cohesive soil is that the void ratio is well above 1 and saturated. Water content is generally high typically about 60% to 90%; close or even higher than the liquid limit. Because of the high void ratio and water content, they are very compressible. However, this type of soil can be improved markedly as they consolidate under a sustained static load. Unfortunately, this improvement by consolidation is accompanied by a volume decrease which may result in an unacceptable ground deformation (settlement). To safeguard against this, the underlying soil is often “force” to consolidate under loads higher than the design loads (termed as “surcharging”) so that the deformations take place prior to final construction of permanent structure.

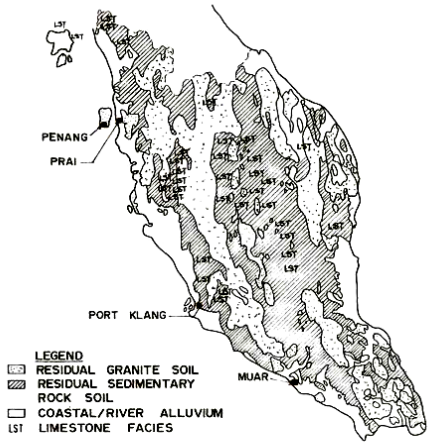

Figure 3. Geological map of Peninsula Malaysia (Ting, 1985)

4.1 Vertical Drains

When the anticipated time for consolidation exceeds the allowable construction schedule, vertical drains are installed to accelerate the rate of consolidation. Vertical drains provide artificial drainage paths for the water flow. Prefabricated vertical wick drain consists of a central core, whose function is to act as a free-draining channel enclosed by a geotextile filter sleeve which prevents the fine soil particles from entering the central core but allows free entry of pore water into the core.

The effectiveness of vertical drains depends on the permeability of the filter and the discharge capacity of the drain. The discharge capacity is significantly reduced when the filter is pressed into the grooves of the central core due to lateral pressure from the surrounding soils and also as a result of ground settlement which causes the drain to buckle or kink. Hence, in very soft soil where large settlement is expected, a more rigid vertical cylindrical drain is preferred. Fig. 4 shows the installation of vertical cylindrical drains for treatment of soft slime in Ipoh for the construction of the North – South Expressway in 1993. It is important to note that the tensile strength of the drain is sufficient so that it will not tear during installation.

Figure 4. Installation of cylindrical vertical drains in Ipoh for the North-South Expressway

Many published papers have been written on vertical drains with surcharge to improve soft cohesive soil for structures, industrial buildings, highways, railways, ports and containers, airports and runways, oil tanks and other infrastructure utilities. One of the earliest reports of a large scale vertical drain project carried out in Malaysia is probably on the coal storage yard of the Port Kelang Power Station Ph. 2 in 1984 (Risseeuw *et al.* 1986).

The 250,000 m^2 storage yard which provides a storage capacity of 760,000 tons of coal with heaps up to 13m height was located on a reclaimed mangrove swamp along the coastline. The soil conditions consisted of about 18.5m of very soft marine clay with 2.5m thick of hydraulic sand placed above. The water content and the compression ratio ($C_c/1+e_o$) was about 60% - 80% and 0.25 from 0 – 7m and 80% - 100% and 0.38 from 7 – 18.5m depth respectively. The undrained shear strength was as low as 10 kN/ m^2 increasingwith depth. In 1984, a total of 3,310,000m of vertical drains was installed to 21m depth at a spacing of 1.41m triangular grid. The criterion of acceptance was an average consolidation of 91% based on a surcharge fill of 10m with anticipated induced settlement of about 3.1m. Fig. 5 shows the undrained shear strength increase. Fig. 6 shows the time-settlement behaviour.

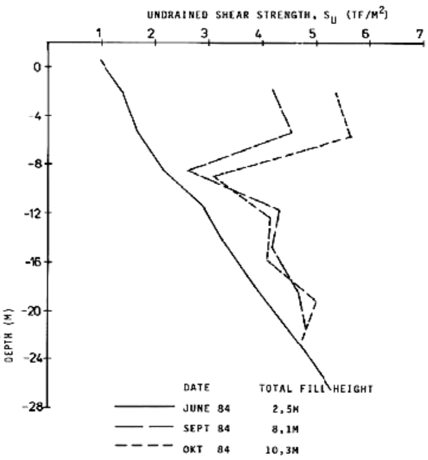

Figure 5. Shear strength increase after consolidation at the coal stor-age yard, Port Kelang Power Station (Risseeuw et al., 1986)

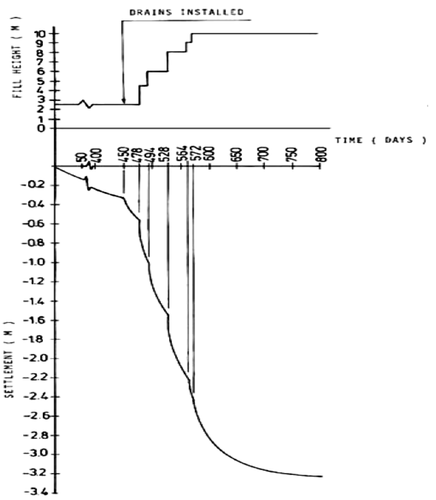

Figure 6. Time-settlement curve at the coal storage yard, Port Kelang Power Station (Risseeuw et al., 1986)

Another early application of vertical drains and surcharge was reported by Wesley & Richards (1987). A field trial was carried out with vertical drains installed at 1.5m and 2m grid to 14m depth at Prai near Butterworth for a housing and light industrial development. It was concluded that due to the complexity of the underlying soil conditions the rate of consolidation and the effectiveness of vertical drains can only be determined by means of a full scale field trial.

There were extensive tin-mining areas in and around Kuala Lumpur (Klang Valley) and Ipoh (Kinta Valley). By mid-1980s, there was a need for rehabilitation of these ex-mining lands and ex-mining ponds for building and road construction. In the rehabilitation process is the treatment of a waste material from mining operations commonly known as slime which is very soft silty clay with fine sand. Slime is weak and compressible. Ting *et al.* (1992) reported compression ratio (Cc/1+eo) ranged from 0.07 to 0.38 with a mean value of 0.2 in Ipoh.

One of the earliest reported housing projects on reclaimed ex-mining ponds is probably the Kampung Pandan Development in Kuala Lumpur (Awang *et al.*, 1987). In 1985, it was decided to reclaim 8 ex-mining ponds with depth of water varied from 3 to 5m and thickness of slime between 8m and 25m. The project was to build walk-up apartments. The ponds were reclaimed using the containment method where the slime and soft clay were trapped beneath a geotextile mat (Yee, 1990). These soft materials were subsequently treated with vertical drains and surcharge. A total of 1,580,000m of vertical drains was installed to maximum depth of 28m at spacing between 1m and 2m. Surcharge up to 5m was placed and settlement induced was about 1m to 1.5m during a period of 4 to 5 months. Soon after the completion of this project, similar housing projects on reclaimed ex-mining ponds such as Kampung Pasir Wardieburn Development at Setapak and Pasar Borong Development at Selayang were adopting this technique to treat slime and soft clays. In 1986 another ex-mining land of 200ha was systematically engineered and developed to the present day resort living water theme park known as Bandar Sunway (Yeow *et al*., 1993; Ooi & Ooi, 2009).

Bridge approach embankments on soft clays are treated with vertical drains and surcharge to reduce post construction settlement and differential settlement between the embankment and the abutment. At the Tinjar Bridge project in Sarawak, vertical drains were installed to depth of 40m. The soil conditions were firm sandy clay at the upper 2m overlying very soft to soft clayey silts/silty clay with N_{SPT} = 0 to 4 down to 40m. Layers of medium stiff to very stiff clay were found below 40 – 45m. Vertical drains were installed at spacing of 1m triangular grid. Settlement up to 1.3m was recorded after 6 months of consolidation. The ground improvement work was completed in 1986 – 87.

Five years later, in 1992 vertical drains were installed to depth of 50m surpassing the previous Asian regional record of deepest drain installation of 45m at Changi Airport (which was carried out in 1979). Fig. 7 shows the 53m vertical drain installation rig at Miri, Sarawak. This is probably the deepest drain installation in Malaysia and among the deepest installation in the world.

With the privatization of infrastructure projects in the late 1980s and 1990s, the application of vertical drains had increased many folds especially with mega projects such as the North -South Expressway, Shah Alam Expressway, Kuantan - Kerteh Railway and the Double Tracking Railway project between Rawang and Ipoh. This has increased the usage of vertical drains to tens of million of metres.

With larger quantity of vertical drains in a single project, the speed of installation played a crucial role in the timely completion of the works and hence, the profitability of the project. Faster and stronger purpose-built installation rigs were developed. Fig. 8 shows the hydraulic installation rig which has an installation capacity of up to 12,000m daily production as compared to 3,000 to 4,000m with the conventional static installation rigs.

Applications of vertical drains have also extended to offshore applications. Vertical drains were required to be installed offshore at the Sapangar Bay Container Port, Sabah. The vertical drains were installed on barges to depths of 20m to 25m. The installation works were completed in 2005

Figure 7. A 50m-vertical drain installation at Miri

4.2 Vacuum Consolidation

Vacuum consolidation was first proposed in the early 1950s (Kjellman, 1952). However, its application has never been satisfactory until Gognon (1991) conducted a full scale research field trial in 1988. The basic procedure consists of installing an air-tight impervious geomembrane over the soft saturated soil to be consolidated (Fig. 9). Vacuum is then created below the geomembrane using a dual venturi air-water pumping system. He demonstrated that the success of vacuum consolidation depends on the ability to create a non-saturated fill layer beneath the impermeable geomembrane in order to maintain a consistent vacuum pressure which acts on the soft cohesive soil. In addition, the detailed construction procedure in creating a complete air-tight seal of the vacuum system around the perimeter of the area to be treated has to be carried out precisely.

Vacuum consolidation is used as a replacement for or supplement to the surcharge fill. Unlike surcharge fill which may cause lateral spreading of the underlying soft soils and pose stability concerns, vacuum consolidation does not pose any stability problem since the treated block of soft soils is "loaded" laterally as well as vertically by the vacuum pressure i.e. vacuum consolidation is isotropic stress increase whereas fill surcharge is deviatoric stress increase. Hence, it is most suitable for very soft soil where stability of construction is of major concern. Principles of the technique are described in Yee *et al.* (2004).

The 1990s saw a rapid development of the vacuum consolidation technology, particularly in countries known as having traditionally very soft compressible soils. Today, an estimated 40 vacuum consolidation projects with more than 6,000,000m^2 has been successfully treated following the above scheme (Fig. 9). The success behind a vacuum consolidation project depends upon a combination of technological know-how and careful implementation of design details. Practical problems such as tears and punctures in the impervious geomembrane, poor seal between the geomembrane and the ground along the peripheral trenches, and vertical drains extending into layers of high hydraulic conductivity (e.g. sand layer) all tend to reduce vacuum efficiency, reduce equivalent surcharge effect and increase pumping capacity and pumping cost of the vacuum consolidation system. Constraining factors such as: (1) adequate horizontal drainage to allow sufficient removal of water which drains out from the soil during consolidation; (2) adequate water saturation along the peripheral trenches, (3) soil stratification including permeable sand seams within the clay deposit at the boundary of treatment, and (4) depth to groundwater, all need to be addressed in a successful implementation of the vacuum consolidation system.

Figure 8. Installation of vertical drains using hydraulic rig.

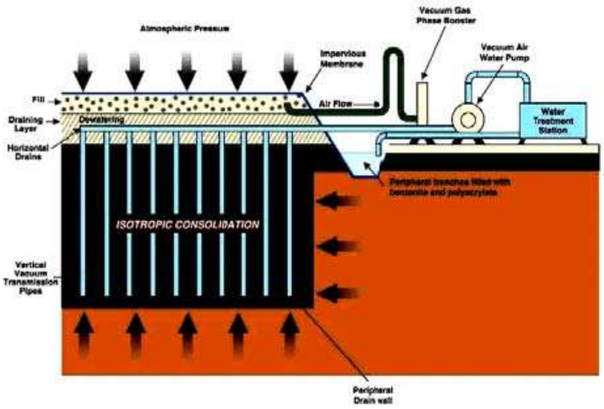

Figure 9. Schematic layout of a vacuum consolidation system

The first country outside France (being the birth place of this scheme) to apply this technique was in Malaysia. It was introduced and used in Malaysia in 1992. Ting *et al.* (1995) reported the first application of vacuum consolidation for the construction of a bridge approach embankment on soft slime and soft clay deposit for the North-South Expressway.

Package 8B-1A involved the construction of a 7m high bridge approach embankment on an ex-mining land located at the southern part of Ipoh. The upper 6m to 12m consisted of slime and soft alluvium overlying limestone formation. The undrained shear strength was as low as 7 – 10kN/m^2.

The bridge structure on piled foundation was constructed ahead of the fill embankment. To construct the 7m high embankment on soft soils, the conventional solution of using vertical drains and surcharge would ideally be the solution. However, due to the close proximity of the piles and the fear of excessive lateral movement of the underlying soft soils upon embankment loading, vacuum consolidation was selected. Furthermore, time was also a constraint for staged construction and consolidation.

The criteria of performance were (i) to maintain a factor of safety not less than 1.3 for the stability of the embankment during construction; and (ii) to limit residual settlement to 10cm over 10 years. The vacuum pressure was maintained at about 0.7 bars which is equivalent to about 3.5m of surcharge fill. This represents a surcharge to embankment height ratio of 0.5. The vacuum pumping was maintained for 3 months.

Construction of the embankment started after two weeks of vacuum pumping. The 7m high embankment was constructed in a single stage without any rest period in-between. The average degree of consolidation with respect to the combined vacuum pressure of 0.70 bars and the 7m high embankment after 3 months of vacuum pumping was about 80%. The average induced settlement was about 70cm. The theoretical factor of safety computed for embankment stability was 0.9 without vacuum consolidation and 1.54 with vacuum consolidation. Lateral movement of the underlying soft soils was limited to about 10mm. Hence, vacuum consolidation had provided the accelerated consolidation required besides, controlling the lateral movement and enhanced embankment stability. It had also reduced the amount of imported fill material for the required 3.5m equivalent surcharge. Fig. 10 shows the close distance of the vacuum treatment area to the installed piles and the bridge structure.

Ooi (1997) and Ooi & Yee (1997) reported the use of vacuum consolidation for the construction of the new Kuching Deepwater Port at Kg. Senari, Sarawak. The deepwater port is located along the Sarawak River and it consists of an island wharf design with 11m water depth to accommodate for 20,000 DWT vessel. The total wharf length is 635m.

The ground condition consisted of an upper 20m of very soft silty clay with shear strength between 10kN/m^2 and 20kN/m^2. Underlying this layer is a layer of soft to firm clayey silt. The water content was about 60% with liquid limit of 70%. The water table fluctuated between 1.5m and 3.5m below working platform elevation.

For the consolidation of the underlying soft clay, vertical drains and surcharge was used for the general container area located some 40m behind the river bank while vacuum consolidation was used along the river bank due to potential instability caused by the surcharge fill as shown in Fig. 11.

At the general area, vertical drains were installed at spacing of 1.5m, 2.0m and 2.5m triangular grid with fill surcharge height of 1.5m and 3m. The vertical drains were installed to 26m depth. Closer to the river bank, vacuum consolidation was carried out. It was maintained at about 0.60 – 0.65 bars which is equivalent to about 3m of surcharge fill. The vacuum pressure was maintained for about 3 months.

The average degree of consolidation with respect to the vacuum pressure of 0.6 bars after 3 months of vacuum pumping was about 75%. The design value was 70%. The induced settlement was about 60cm. At the general area, for an “equivalent” area with 1.5m grid vertical drains and 3m surcharge, the settlement after 12 months was about 50cm while the control area without vertical drain, the settlement was 33cm. Field vane shear tests were carried out at the same locations before and after vacuum consolidation. The average increase in shear strength was about 14kN/m^2 as compared with the theoretical value of 12kN/m^2.

In this project, vacuum consolidation has provided the necessary consolidation in a shorter time and the necessary stability required during the works. Without vacuum consolidation, the consolidation works would have been difficult, if not impossible to achieve with problems of instability to the river bank. The vacuum consolidation works was completed in 1996. Fig. 12 shows the vacuum treatment area beside the Sarawak River.

As described earlier, the intrigue characteristic of a successful vacuum consolidation lies on the effectiveness of the system in creating the necessary isotropic consolidation state. This involves the detailed implementation of the necessary works in creating an air-tight sealing system as well as the maintenance of a consistent vacuum pressure through the non-saturated fill layer beneath the geomembrane. Unsuccessful application has been reported where the above characteristic and careful implementation of works according to details were not followed.

Tan & Liew (2000) reported a failed embankment on vacuum treated area during construction. The embankment was constructed on very soft silty clay of 4.5m thick overlying soft sandy clay to depth of 12m. A layer of very loose clayey sand was found below the sandy clay. Undrained shear strength varied from 10kN/m^2 to about 20kN/m^2. Cracks appeared when the embankment reached about 5m height in about 110 days. The pore water pressure measurements were taken. It showed a trend of increasing pore water pressures for more than one month during the construction. Vacuum pressure was insufficient.

Figure 10. Vacuum consolidation for the North-South Expressway in Ipoh

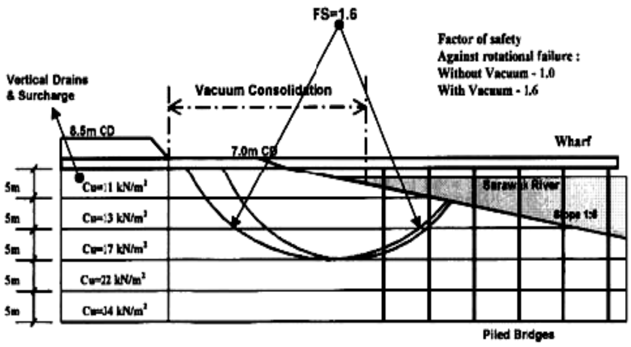

Figure 11. Simplified cross-section of ground improvement scheme for the New Kuching Deepwater Port

Figure 12. Vacuum consolidation for the New Kuching Deepwater Port at Kg. Senari, Sarawak

4.3 Suitable Types of Soil for Consolidation Treatment

The type of soil most suitable for consolidation by vertical drains (in combination with fill surcharging) and vacuum consolidation is normally to slightly overconsolidated saturated soils with low permeability such as soft clays and silts, and slime. The greatest effectiveness is in inorganic clays and silts that exhibit little secondary compression since vertical drains do not affect the rate of secondary compression. To minimize the effect of secondary compression, additional surcharge or extending the surcharge period is necessary. Cognon *et al*. (1994) reported the applications of vacuum consolidation for a road embankment on 3.7m thick of peat and organic clay with water content ranging from 400% to 900% for the peat and 140% to 210% for the organic clay.

4.4 Design Issues

The design of a surcharging program involves the computation of (1) the time-settlement curve under the design load; and (2) the time-settlement curve under the surcharge load. The classical one-dimensional consolidation theory is used. To determine the time required for surcharging, one has to determine the estimated total settlement (or the required induced settlement after considering the allowable residual settlement) on the time-settlement curve of the design load. Then, from the time-settlement curve of the surcharge load one has to predict the time of surcharging which would result in the same amount of induced settlement required.

If the time required for surcharging is more than the permitted time (which is often the case), vertical drains are installed to accelerate the consolidation process. In this case, both radial and vertical drainage are considered in establishing the time-settlement curve of the surcharge load. The theory of consolidation by radial and vertical drainage is well established (Barron, 1948). The design procedure for vertical drains is described in Hansbo (1979).

Although the mechanism of consolidation between vertical drains with surcharge fill and vacuum consolidation may be different, the results are rather similar. In the essence, geotechnical design analyses used to evaluate vertical drain spacing, settlement rate and strength gain for surcharge fill with vertical drains are equally applicable to vacuum consolidation. However, the stablilty analysis is different. Surcharge fill is basically a deviatoric stress increase while vacuum consolidation is isotropic stress increase without the risk of instability as in the case of the deviatoric stress increase.

4.5 Performance Evaluation

Among the common geotechnical instruments used in a consolidation project are settlement plates, piezometers and inclinometers to measure and monitor for ground deformations and the build-up and dissipation of pore water pressures with time. These measurements are used to determine the placement and removal of surcharge fill as well as to control stability during construction.

4.6 Choice of Consolidation Methods and Selection Criteria

The rate of consolidation is affected by (1) the available drainage facilities; and (2) the rate of filling (e.g. in embankment construction). By reducing the vertical drain spacing, it increases the rate of consolidation. However, the rate of filling is not affected by the presence of vertical drains but solely controlled by the shear strength of the soft clay. Vertical drains serve no structural support to the soft clay.

Typical undrained shear strength (c_u) of soft clays can be as low as 5kN/m^2 to 20kN/m^2. Due to such low strength, surface loading of soft clay would anticipate progressive failure. When the stress imposed by a load such as an embankment exceeds the strength of the soft clay foundation, bearing capacity or "mud-wave" failure takes place. Because soft clay is also somewhat sensitive in most cases, bearing failure often takes place at a much lower stress than is calculated by the general shear failure bearing capacity analyses. Conventional bearing capacity analyses for an embankment indicate an ultimate capacity of q_o = 5.14c_u. However, analyses assuming elastic conditions up to the instant of local soil shearing show that failure could develop progressively when the stress reaches q_o = 3.14c_u. Therefore, at any stage of construction the height of filling is limited. In fact, in vertical drains and fill surcharge there is a basic contradiction in that the soft soil which cannot sustain the normal imposed embankment load is now called upon to support additional surcharge load. Pressure berms may be used to provide stability when the height of fill is high. However, in most cases there is space constraint. Stage construction can be considered to take advantage of the increase in the shear strength under the imposed load at each stage of fill placement. Again, in most cases sufficient time is not available. Hence, for construction on "ultra" soft soil where stability is of major concern and time is limited, vacuum consolidation may be the solution to the problem.

5. DENSIFICATION METHODS

Deposits of very loose granular soil or cohesionless soil (e.g. sand) require improvement. They are somewhat "compressible" (Varaksin & Yee, 2007) and are very unstable when subjected to even a modest shock and vibration. D'Appolonia (1970) reported that granular soil is prone to liquefaction. For small strain of the order of 10^{-5} to 10^{-3} the minimum relative density to prevent liquefaction should be about 70% and that fine sand with a relative density less than 50% is subject to liquefaction during ground motions with acceleration in excess of 0.1g.

Deposits of natural loose sand are found in coastal areas. Similarly, loose sand mingled with clayey material is found in riverine deposit generally within a watercourse that has been repeatedly deposited in times of flood and recession of waters.

In recent years, coastal reclamation presents a major source of loose sand. Much land has been reclaimed using hydraulically filled sand. The large volume of water needed in hydraulic filling must be "ponded" to allow sufficient time for the sand to settle. The resulting structure is likely to be very loose and it will remain loose and saturated because of the capillary retention of the sand which prevents the sand particles from rolling into a stable and denser orientation. Relative density between 40% and 80% after hydraulic placement was reported by Choa & Bawajee (2002) in the Changi Reclamation project. Values of N_{SPT} can be as low as 3 to 4 while CPT cone resistance q_c can be as low as 0.5 to 3MPa were measured below mean sea level.

Mine tailings from tin mining operation consist of fine loose sand. Loose sand tends to deposit nearer to the discharge point of the tailings due to its weight as compared with slime which tends to deposit further away from the discharge point. For rehabilitation of ex-mining lands, it is necessary to improve the loose sand tailings.

While loose sand is not as compressible as soft clay, the compressibility is sufficiently great that it cannot be ignored in the design of foundation. More important, is the inherent instability of the loose sand particle orientation. Although loose sand may be unstable and change state readily, their very instability nature makes it possible to alter their structure effectively. Vibration through shearing of the loose sand particles to form a denser and stable orientation has been the most effective means for densifying loose sand. This results in higher bearing capacity, lower settlement and increased resistance to liquefaction. At the surface, densification is accomplished by surface compaction. When at depth, densification is more difficult. It requires special technique and equipment.

5.1 Dynamic Compaction

This process is also known as dynamic consolidation or heavy tamping. This is one of the most versatile and least expensive ground improvement techniques. The densification by dynamic compaction is done systematically usually in a pre-determined grid pattern. It consists of delivering high energy impacts at the ground surface by repeatedly dropping steel pounders, 10 to 40 tons from heights ranging from 10 to 40m as shown in Fig. 13. The spacing between the impact points depends mainly on the depth of treatment, the grain size distribution and its permeability and the location of the ground water level. Deep craters up to 2m are formed upon impact. The craters are filled with sand after each pass. In loose sand, the heave around the craters is generally small.

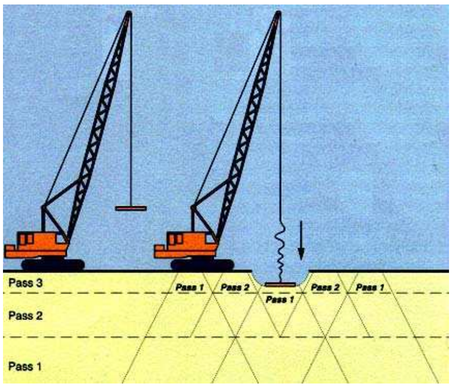

Figure 13. Various phases of dynamic compaction process

The initial spacing of the impact points usually corresponds to the treatment depth. It is often advantageous to use maximum compaction energy (with heaviest pounder falling from maximum drop height) for the first few blows in order to extend the compaction effect as deep as possible. The spacing is reduced for the subsequent passes thereby allowing adequate compaction to be carried out at the shallower depth.

The depth of improvement is related to the compaction energy per blow. Dropping a 15-ton pounder from 20m will give 300 ton.m compaction energy per blow. Figure 14 shows the depth of treatment against the compaction energy.

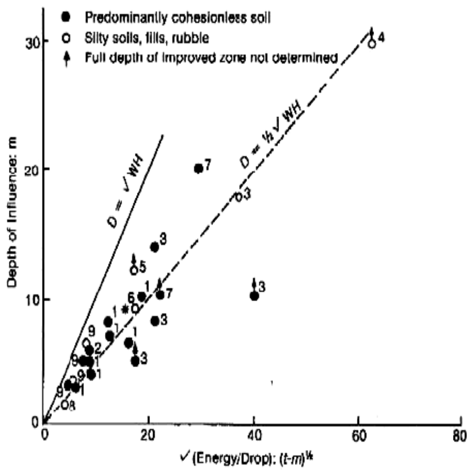

Figure 14. Compaction energy against depth of influence / treatment depth (Green & Kirsch, 1983)

Compaction is generally higher below the pounder. Maximum increase in density is at about one third of the depth of treatment from the surface. An increase of the penetration resistance of 300% to 400% can be expected in sand and gravel. In marginal sand and unsaturated fill materials, the increase is usually about 200 to 300%. Relatively large area greater than 15,000m^2 have to be treated in order to increase its cost-effectiveness due to higher cost for mobilization. Typical production rate is about 12,000m^2 to 15,000m^2 per month using one rig working on a single shift.

One of the earliest published applications of ground improvement in Malaysia is dynamic compaction for a housing development project in Kuala Lumpur in 1978 (Ting, 1982 & Ting *et al.*, 1982). The site was a valley used as a soil dump. It was filled with materials ranging from boulders to cobbles, gravels, sand, silt and clay without any compaction. The thickness of fill was about 12m. The material was probably partially saturated. Hence, settlement is expected with saturation by infiltration of water over a period of time. Thus, settlement was a problem although bearing capacity may be adequate. Dynamic compaction was carried out using a 13.5 ton pounder with a drop height of 25m. Compaction energy was about 337 ton.m per blow and it was applied over 5 phases at grid spacing of 7m. Total compaction energy used ranges between 135 – 270 ton.m/m^2. The enforced settlement was between 40 – 96cm. Results from the pressuremeter tests show improvement down to 12m depth. The mean limit pressure (P_L) was increased from 5 bars to 13 bars and the mean pressuremeter modulus (E_P) was increased from 60 bars to 150 bars after treatment. The improvement was about 250% to 300%.

Toh *et al.* (1985) and Ooi (2007) reported the application of dynamic compaction for the foundation of Wisma Saberkas in Kuching. It is a 22-storey tower block resting on a raft foundation; a surrounding 7-storey podium founded on strip foundation and a 7-storey car park on steel H-piles. The raft and the strip foundations rest in part on rock and in part on soil which has been improved by dynamic compaction. The soil condition was clayey silty sand overlying loose sand over very hard sound unweathered sandstone rock at depth of 4m to 9m below ground surface. Dynamic compaction was carried out using a 13.5 ton pounder with a drop height of 25m over 2 phases at a grid spacing of 6m. Altogether, 16 blows were delivered giving a total compaction energy of 150 ton.m per m^2. Before commencement of dynamic compaction, the area was first covered with sand as working platform. After dynamic compaction, the craters were backfilled with sand and the second phase of dynamic compaction was carried out in-between the prints of the first phase. The craters were similarly backfilled with sand. Finally, the entire area was then given the ironing phase with a lower drop height on an overlapping compaction grid pattern.

Maximum settlement was computed to be about 43mm where the soil was deepest with the least stiffness. Maximum angular distortion was about 1/470. Settlement measurement taken after construction agreed well with the calculated values. The compaction work was completed in early 1983.

With increasing demand, more housing developments were carried out on filled ground over valleys. Most of these areas are non-engineered fill. Uncompacted fill of various compositions ranging from boulders and rock pieces, construction debris to silt and clay were used to fill valleys without compaction. Hence, dynamic compaction was used to improve bearing capacity and reduce post construction settlement.

Fig. 15 shows dynamic compaction carried out for a housing project at Bandar Menjalara for double storey link houses on individual footings. The fill material was a mixture of residual soil with rock and boulders. Anticipated problems were settlement due to self-bearing and collapse settlement. The enforced settlement was 40 – 60cm with treatment depth between 5 – 7m.

Figure 15. Dynamic compaction at Bandar Menjalara for double storey houses on non-engineered filled ground.

Similar treatment method was adopted by established developers at Kepong, Puchong, Desa Sri Hartamas, Hulu Langat, Jalan Kelang Lama areas, etc. for their housing projects on non-engineered fill ground over ponds and valleys. In certain cases, dynamic compaction was carried out close to built-up housing units at a distance of 10 – 15m away. In such cases, vibration monitoring was carried out. Where the peak particle velocity exceeds the permissible value of 8mm/s, an open trench was dug to absorb the surface wave energy.

The heaviest compaction on filled ground was carried out in 2005 for a housing development at Desa Sri Hartamas, Kuala Lumpur where the thickness of non-engineered fill extends to more than 15m. A 23-ton pounder dropping from 20m delivering compaction energy up to 460 ton.m per blow was used to compact fill ground with large sized boulders. Two-storey semi-detached houses were constructed on individual footings after treatment.

In 1994 – 95, dynamic compaction was carried out using the 750 ton.m capacity Hecto machine (being considered as the 3rd largest compaction machine in the world) to lift 25 tons pounder to 30m drop height for the Shah Alam Expressway (Fig. 16). This represents the largest compaction rig used in Malaysian history.

Figure 16. Hecto 750-ton.m DC rig used in Shah Alam Expressway

In 2000, shallow vibratory compaction (SVC) was introduced for the first time for the Kuantan – Kerteh Railway project. The objective was to densify the upper 3 – 4m loose sand overlying cohesive soil. In the Double Tracking Railway project between Rawang and Ipoh, more SVC works were carried out. Fig. 17 shows SVC works using dynamic compaction at close distance to existing "live" track. Typical CPT result is as shown in Fig. 18 before and after treatment.

Figure 17. SVC using dynamic compaction at Bidor-Kampar for the Rawang – Ipoh Double Tracking project

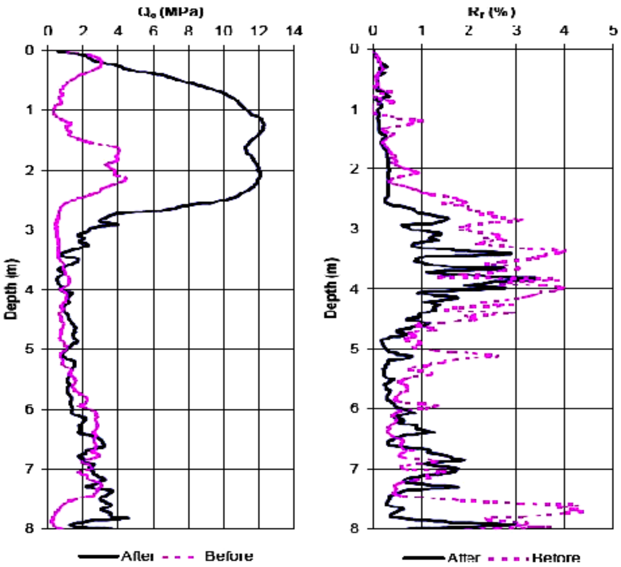

Figure 18. CPT results before and after SVC at Bidor - Kampar

5.2 Vibro Compaction

Vibro compaction involves a process of re-arrangement by shearing of soil particles into a denser orientation by means of horizontal vibration. Vibrations are created in a horizontal plane providing a lateral compaction effort. Typical equipment used includes an electric or hydraulic vibroflot suspending from a crane as shown in Fig. 19. The vibroflot consists of a torpedo shaped horizontally vibrating probe that vibrates at frequency of 30 to 50Hz with an amplitude of 10 to 40mm. Fig. 20 shows the vibro compaction process.

Spacing of compaction points depends upon the soil type, density requirements and the vibroflot characteristics. It relied on personal experience of the engineers and contractors and semi-empirical design charts. Recent research has indicated that the degree of improvement is also a function of the initial density and clay fraction in the soil, among other parameters. The final field compaction spacing is usually established with the completion of a field trial compaction at the start of the work. Typical spacing for the compaction points ranges from 1.5m to 4.5m.

Figure 19. Electric vibroflot for sand compaction

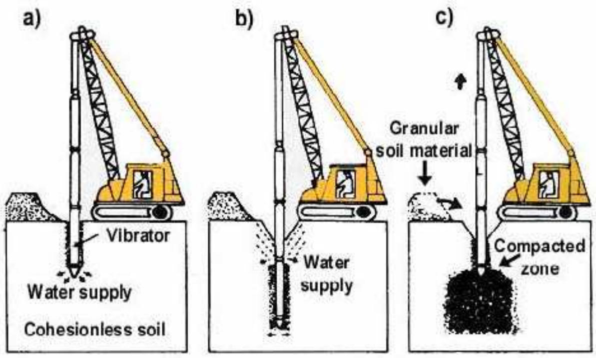

Figure 20. Vibro compaction process

The compaction is generally higher closer to the vibrator. The relative density is usually about 100% up to 0.3m to 0.5m from the vibroflot. The compaction decreases gradually with increasing distance. The lowest relative density is usually obtained half-way between the compaction points. An increase of the penetration resistance of about 200% to 300% can be expected in clean sand. The typical production rate is about 5,000m^2 to 7,000m^2 per month using one rig working on a single shift.

Vibro compaction was used at the Bidor – Kampar package of the Rawang – Ipoh Double Tracking Railway project (Fig. 19). It was used at close distance of less than 10m to the existing "live" railway track where dynamic compaction produced surface vibration more than the allowable value of 14 mm/sec.

5.3 Suitable Types of Soil for Compaction Treatment

Dynamic compaction was developed into a "systematic" means of densification of loose, cohesionless saturated soil by Louis Menard in the late 1950s (Menard & Broise, 1975). Upon impact, the saturated soil liquefies and the densification process is induced by shearing of the soil particles into a denser orientation. Vibro compaction has similar densification mechanism but has lesser compaction energy compared with dynamic compaction.

Vibro compaction is best suited for use in clean sand with percentage of fines (particles less than 63μm) generally not exceeding 10% and clay content not exceeding 2 – 3%. The effectiveness for vibro compaction reduces in clayey and silty soil. It is also less effective in very fine uniformly graded sand. The effectiveness of vibro compaction is also reduced in cemented sand. The reason being the cohesion provided by these fine materials prevents the momentary breaking of friction bond between particles through vibration and in saturated soil the lower permeability impedes the densification process. Hence, the application of vibro compaction in Malaysia is not as widely used compared with dynamic compaction due to the fines content. In most cases, loose soil is marginally clean sand (e.g. silty or clayey sand). Also, most non-engineered fill areas contain large aggregates or boulders. In such conditions, dynamic compaction is more suitable as penetration of the vibroflot may be difficult and may cause damage to the equipment. Dynamic compaction has been successfully used for soil containing higher fines content but in most cases, the soil is non-saturated. Although, it has been used in saturated cohesive soil, its success is uncertain and may require special attention to the generation and dissipation of excess pore water pressures. For saturated soil, a limiting fines content of about 20 - 25% applies to dynamic compaction.

Although dynamic compaction is conventionally used for densification of loose sand, the majority of the dynamic compaction works in Malaysia in recent years has been performed at sites of non-engineered fill, ex-mining land, solid wastes and landfill sites (municipal wastes). Dynamic compaction was carried out at the Jelutong landfill site for the Jelutong Sewerage Treatment Plant in Penang. Another increasingly common application has been for the stabilization of collapsible soil which is usually stiff and dry in their natural state, but lose strength and experience significant "collapse" settlement when they become wet, and also ground with shallow cavities and metastable in nature. The Batu Toll area of the DUKE Expressway, located in mined out land over the cavernous limestone bedrock was treated with heavy dynamic compaction for this reason.

5.4 Design Issues

The design of dynamic compaction requires the determination of pounder weight and size, grid pattern, drop height, number of blows and phases and the depth of influence. Eq. 1 can be used to determine the depth of influence (D):

$$D = n\sqrt{W.H} \tag{1}$$

where W is the pounder weight (tons); H is the drop height (m) and n is the soil type rheological factor which varies between 0.3 and 0.8 depending on the type of soil.

The grid spacing is related to the impact energy as denoted by Eq. 2:

$$E_a = \frac{N.W.H.P}{S^2} \tag{2}$$

where E_a is the average applied energy (ton.m) over the treated area; N is the number of blows; W is the pounder weight (ton); H is the drop height (m); P is the number of passes and S is the grid spacing (m).

The design of vibro compaction involves selection of grid pattern, spacing and depth. These parameters depend on the types of soil, the required densification and the characteristics of the vibroflot used. Based on past experience, semi-empirical design charts are produced according to the characteristics of the vibroflot. A larger capacity vibroflot with a bigger centrifugal force, larger amplitude and a lower frequency will usually has a larger grid spacing of compaction points compared with a smaller capacity vibroflot. Fig. 21 shows an area pattern design chart for vibro compaction (Glover, 1982). The actual spacing of compaction will be confirmed with field trial compaction results.

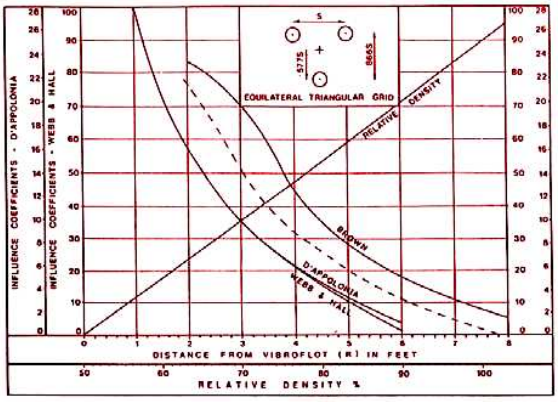

Figure 21. Area pattern design chart (Glover, 1982)

5.5 Performance Evaluation

For dynamic compaction, the performance indicators are the pounder penetration depth, crater volume and size, amount of ground heave and subsidence and the amount of backfilling material. Cone penetration tests (CPT) and pressuremeter tests (PMT) are often carried out to assess the effectiveness of dynamic compaction. When there is a nearby structure or utility, vibration monitoring is necessary.

For vibro compaction, the degree of densification in terms of relative density is usually specified. Relative density is often used as an intermediate soil parameter. This is usually measured by CPT q_c. Direct derivation from q_c of relative density (as well as the angle of shearing and modulus values) depends on empirical correlations. These may have some backing from calibration chamber tests, but it should always be remembered that such correlations are limited in the range of soil to which they apply (Meigh, 1987). Alternatively, PMT may provide a better in-situ test to measure the effectiveness of vibro compaction in terms of limit pressure for bearing capacity calculation and pressuremeter modulus for settlement calculation. However, PMT is time consuming to carry out. It is suggested that a combination of CPT and PMT is to be performed with probably 1 PMT for every 3 CPT. Evaluation of improved ground is usually done at locations intermediate between compaction points.

5.6 Choice of Compaction Methods and Selection Criteria

The determinative factor in the choice of compaction method is the type of soil or fill material to be compacted. Beside, environmental constraint needs to be considered. Because of the inherent characteristics of a heavy pounder hitting the ground, dynamic compaction produces vibration concerns. The acceptable vibration limits vary from one standard to another. According to the German Standard DIN 4150 peak particle velocity (PPV) less than 8mm/sec will not likely to cause any damage to adjacent structures supported on spread footings due to settlements of the underlying materials. Structural damage requires a much higher PPV up to 50mm/sec. The PPV at a distance of 30m from the point of impact is usually less than 50mm/sec. To reduce vibration, a cut-off trench of 1.5 – 2m deep is usually constructed to intercept the surface wave which causes the vibrations. Yee & Ooi (2003) reported a close distance of 7m from a service train line where dynamic compaction was carried out using a 15ton pounder dropping from 20m. With trenching, the measured PPV was limited to 15mm/sec at a distance of 7m. Fig. 22 shows the PPV measured at Bidor – Kampar site for dynamic compaction plotted together with measured PPV for vibro compaction.

Dynamic compaction is cost-effective for large compaction areas (> 15,000m^2) with its high productivity but only if the surface vibration is not an issue of concern. Vibro compaction is cost effective when the require treatment depth exceeds 10 – 15m but for sand having less than 10% fines. With increasing fines content such as marginal dirty sand, the spacing of compaction starts to be close and productivity drops substantially.

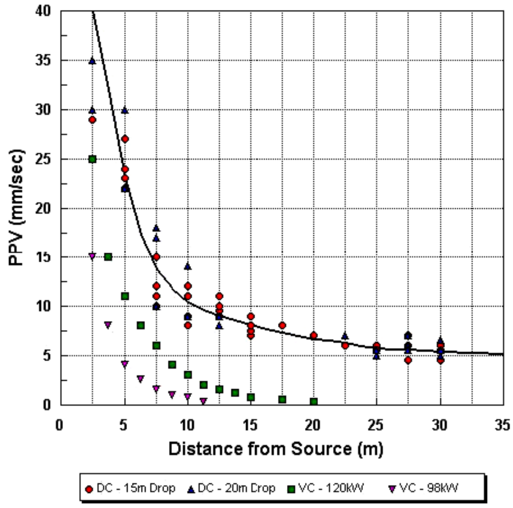

Figure 22. PPV for dynamic compaction (DC) using 15-ton pounder and vibro compaction (VC)

6. REINFORCEMENT METHODS

Experience has shown that silty clayey soil does not react to compaction effectively. Fig. 23 provides an indication of the soil compactibility in terms of CPT cone resistance (q_c) and friction ratio (R_f) results. For this type of soil, the improvement is usually measured by the percentage of the soil replaced.

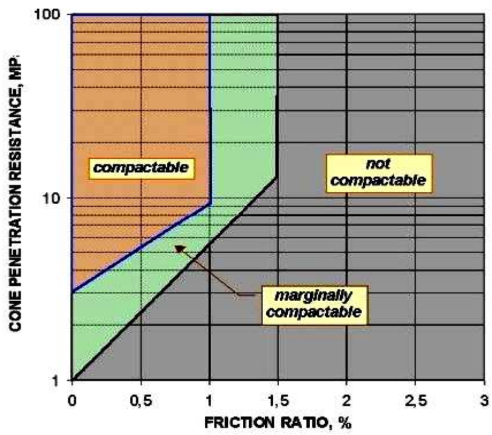

Figure 23. Soil compactibility according to CPT results (Massarch & Heppel, 1991)

In-situ reinforcement of poor soil is accomplished by inclusion of vertical reinforcing elements in the soil with the main benefit resulting from the structural aspect of these elements. These elements can be non-rigid inclusion (e.g. stone column) or semi-rigid inclusion (controlled modulus column). Reinforced concrete piles are rigid inclusions.

The non-rigid inclusions operate as stiff but compressible columns embedded in weaker soil. The applied load is distributed by a granular layer (usually sand) acting as a “flexible raft” on top of the inclusions. These inclusions act as a group to support the distributed load. The soil and its reinforcing elements act in combination, interacting through friction and adhesion to increase the shear strength of the soil mass; to reduce its settlement under the load and to improve its resistance to liquefaction. The volume of soil replaced by these reinforcing elements is referred to as the area replacement ratio (A_{col}/A) where A_{col} is the area of the reinforcing elements and A is the total influence area. Typical area replacement ratio for non-rigid inclusions is between 15% and 30% and 2% and 8% for semi-rigid inclusions.

6.1 Vibro Replacement/Displacement Columns (Stone Columns)

Although constructed using the same equipment and work procedure as vibro compaction (except that the vibroflot has a smaller amplitude and higher frequency), stone columns function as reinforcement rather than densification. They are used in soft soil to (1) increase bearing capacity; (2) reduce settlement; (3) accelerate the rate of consolidation; (4) improve stability; and (5) resist liquefaction. It involves replacing 15 – 30% of the cohesive soil with stones in the form of columns in most applications.

Typical diameters of stone columns are between 80cm and 100cm and the column spacings are between 1.6m and 2.5m. Stone columns are installed using the wet method (vibro replacement) or the dry method (vibro displacement). Other methods such as rammed columns using temporary casing have also been used but to a lesser extent.

The vibroflot penetrates the ground under its own weight aided by water jetting as in the wet method (Fig. 24) or compressed air as in the dry method (Fig. 25). Horizontal vibration is produced close to the base of the vibroflot. Stone backfill is introduced in control lifts, either from the surface down the annulus created by penetration of the vibroflot as in the top feed wet system or through feeder tubes directed to the tip of the vibroflot as in the bottom feed dry system.

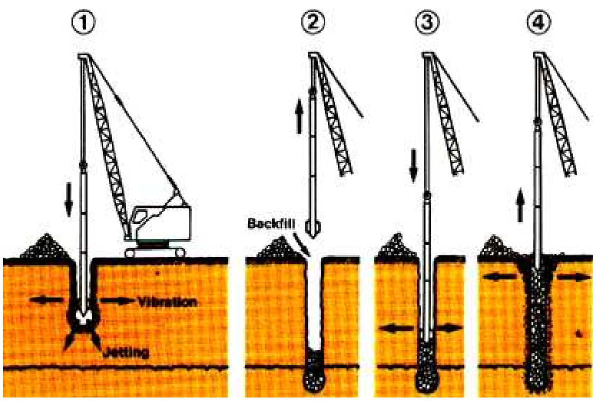

Figure 24. Installation of stone columns by top feed wet method

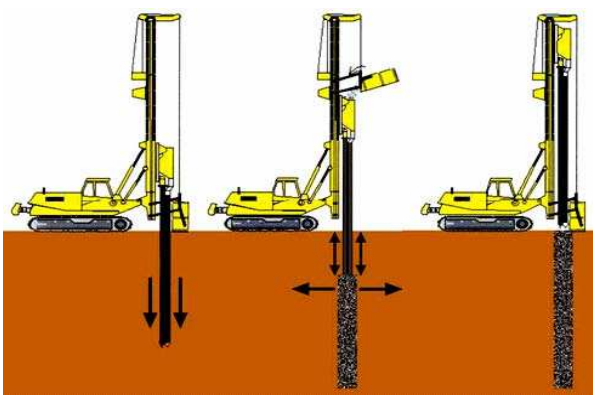

Figure 25. Installation of stone columns by bottom feed dry method

The wet method is generally used where the "borehole" stability is questionable. Hence, it is suited for sites underlain by very soft to firm soils and a high ground water table. Whereas, the dry method is suited for firmer soils with a relatively low ground water table (FHWAA, 1983). However, the main controlling factor is the availability of a nearby source of water for wet method. Otherwise, it would be the dry method.

Stone columns were installed using the bottom feed dry method for the Kajang Ring Road and the DUKE Expressway. For the Kajang Ring Road project, the soil condition at Serdang site was soft to firm clay/silt with N_{SPT} varied from 2 to 5 down to 10m depth. Below 10 – 12m, stiff clayey silt with $N_{SPT} > 15$ was encountered. Stone columns of 70cm diameter were installed to depth of 10 – 12m at triangular spacing of 1.6m to support a 9m high road embankment. The installation works was completed in 2000.

For the DUKE expressway project, bottom feed dry method was used at Setapak site where stone columns were installed to 22m deep. At the Jalan Kuching site, stone columns were installed to 12m and at the Segambut site, stone columns were installed to 18m. The stone columns were 100cm diameter and installed at spacing between 1.6m and 2.3m. With greater depth of installation, a crawler crane-based machine was utilized. Fig. 26 shows the SAS rig of 35m section used in the South Klang Valley Expressway (SKVE) which is considered as one of the largest rigs for stone column dry method installation in Malaysia. Total quantity of stone columns installed in the SKVE expressway exceeded 400,000m.

Figure 26. Dry bottom feed stone column installation at SKVE

While bottom feed dry method was used due to non-availability of water, top feed wet method is usually used for higher productivity and generally larger column size. Top feed wet method was used to install 100cm diameter stone columns to 12m depth at Kepong for DBKL (Kuala Lumpur City Hall) road project. The stone columns are used to support a reinforced earth supported bridge approach embankment of 6m height founded on soft silty clay with N_{SPT} between 1 and 4. The columns were spaced at 1.7m, 2m and 2.3m. Plate bearing tests on single stone columns were conducted and plate settlement of 6 – 9mm was recorded at 150% of the design load.

Similarly, top feed wet method was used at Sabak Bernam (Selangor) for a JKR (PWD) road project. The road embankment was 4.3m high with 1.5m surcharge founded on very soft marine clay with N_{SPT} between 0 and 2. Small stone columns of 60cm diameter were installed at 1.2m, 1.5m and

1.8m triangular grid down to depth of 13m. These stone columns may well be the smallest diameter stone columns ever installed using wet method. The design of these small diameter stone columns was to act as vertical drainage columns and at the same time increase the bearing capacity of the ground. The stone columns were tested using plate bearing test. Plate settlements of 8 – 15mm were recorded under a load of 150kN/m^2. Total quantity of stone columns installed was about 70,000m. The installation works was completed in 2005.

6.2 Dynamic Replacement Columns

Similar to vibro replacement (stone columns) which is an extension of vibro compaction, dynamic replacement (DR) is an extension of the dynamic compaction. Although constructed using the same equipment but with a different work procedure and a different pounder shape, dynamic replacement is more a reinforcement rather than densification. Dynamic replacement is applied to soft cohesive soil to (1) increase bearing capacity; (2) reduce settlement; (3) accelerate the rate of consolidation; (4) improve stability; and (5) resist liquefaction.

This technique starts out by producing a pilot DR crater (“print”) with light pounding. Unlike vibro stone columns where only stones of certain sizes are used, the DR craters can be backfilled with sand, aggregate, stone or even rock pieces (up to 300mm size) or a mixture of these materials that will lock together under subsequent heavy pounding. Because of the higher permeability of these backfill materials, pore water pressure from the underlying and adjacent cohesive soil will dissipate quickly. This process is repeated until a noticeable decrease in crater formation occurs. The dynamic replacement process is shown in Fig. 27.

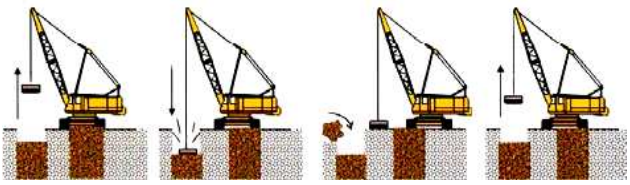

Figure 27. Columns formed by dynamic replacement (DR) process

This technique results in large diameter columns of compacted granular backfill material. Typical diameter up to 2.5 – 3m is common for dynamic replacement. The surface area of these columns is approximately 5m^2 compared with 0.8m^2 for a 100-cm diameter vibro stone column. Hence, a working load up to 80 – 100 tons per column is typical for dynamic replacement columns in soft to medium stiff clays compared with 20 – 25 tons for vibro stone columns. Similar to vibro stone columns, these columns also act as large vertical drains while providing structural support.

The concept of dynamic replacement is similar to vibro replacement. It involves typical replacement ratio of 15 – 30%. Typical spacing of columns varies from 4.5m to 7m and a typical configuration of a dynamic replacement scheme would be a column of 2.5m diameter at 5m square grid. This will give a replacement ratio of 20%. This is equivalent to vibro stone column of 1m diameter at 2m square grid. Besides having a larger influence area, dynamic replacement columns do not bulge readily upon loading due to its larger sectional area and thus, better bearing capacity. Since the DR columns are installed by heavy pounding, any localized “bulging” (due to localized soft layers) would have been induced by the numerous high energy impacts during installation of columns prior to permanent loading.

Dynamic replacement started about 10 years later after the introduction of dynamic compaction in Malaysia. One of the earliest published applications of dynamic replacement in Malaysia most probably is the Medan Pejasa housing development project at Jalan Kelang Lama (Ali *et al*., 1997) and the Kampung Pakar housing development at Sg. Besi (Lee *et al*., 1989). Both projects were completed in 1988.

The Medan Pejasa project site was located within a disused ex-mining land comprised of an ex-rubbish dumping site and an ex-mining pond. The ground condition was 6 – 7m thick rubbish (household wastes) overlying layers of loose silty clayey sand and clayey silt. The ground water table was about 2m below surface. Single and double-storey terrace link-houses were to be constructed. The acceptance criteria for ground improvement works were (1) a safe bearing capacity of 120kN/m^2; and (ii) a maximum differential settlement of 1/600. The improvement works consisted of excavation and replacement of the upper 2.5 – 3m young rubbish and backfilled with clean sand. Dynamic replacement columns were installed below individual footings and dynamic compaction was carried out everywhere else. A 2msurcharge was placed for 6 – 8 weeks for settlement monitoring.

Dynamic replacement was carried out with a 14-ton pounder dropped from 20m over minimum 3 phases of compaction. At the structural areas, 24 blows per print were delivered giving total compaction energy of 240 ton.m per m^2. At the infrastructural areas, 16 blows were delivered giving total compaction energy of 190 ton.m per m^2. The enforced settlement obtained during the compaction works was about 60cm which represents about 14% of the remaining rubbish thickness. In-situ pressuremeter tests were carried out. The limit pressure had increased from 3 – 4 bars to 12 – 16 bars after treatment. The pressuremeter modulus had increased from 25 – 30 bars to 80 – 140 bars. During the 2msurcharging period, the measured average settlement was about 13mm.

The Kg. Pakar development consists of 8 blocks of 5-storey apartments built on an ex-mining land. The ground conditions were highly heterogeneous and consisted of loose sand and silt with soft clay pockets. The N_{SPT} varied from 0 to 2 for the soft clay and 2 to 10 for the loose sand. All the apartments are on raft foundation after ground improvement using a combination of vertical drains and surcharge, dynamic compaction and dynamic replacement. For dynamic replacement, a 15-ton pounder was used. The maximum drop height was 25m. Compaction energy up to 270 – 335 ton.m per m^2 was delivered. The enforced settlement was about 60cm to 85cm. The improvement depth was about 8 – 9m.

During the period of rapid development in the infrastructure sector, dynamic replacement was used extensively for the construction of road and railway embankments where marginal soil was shallow extending down to depth of 5 – 6m.

Dynamic replacement columns using rockfill material was used on an ex-mining land (slime) for the Ipoh to Gopeng package of the North-South Expressway in 1993. Similarly, DUKE Expressway at the Jalan Kuching site was treated with dynamic replacement “rock” columns in 2006. Dynamic replacement “sand” columns were used for the Shah Alam Expressway and the Ipoh-Lumut Expressway in 1994 – 95. Yee & Ting (2004) reported the use of combined dynamic replacement “sand and stone” columns with vertical drains for the construction of a road embankment up to 16m height on soft clay and peat at Putrajaya U4 Interchange in 2001. Similarly, the semi-high speed test track for PROTON (Malaysian national car manufacturer) at Shah Alam was treated with a combination of dynamic replacement and vertical drains in 1993 – 94. Upgrading of existing roads at Puchong and Putrajaya in 2003 and the construction of new by-pass at Machang in 2005 were constructed on dynamic replacement “sand” columns. An estimated area treated with dynamic replacement for roads exceeded 1,000,000m^2 by end of 2006.

Railway construction presents a major application for dynamic replacement in the early 2000s. Some of these railway projects include the Petronas Railway between Kerteh and Kuantan in 2000 – 01 and the Double Tracking Railway between Rawang and Ipoh in 2001 – 04. The Rail Link between Senai and Tg. Pelepas port was constructed on very soft organic silty clay down to 5m depth with 7 – 12% organic content. Fig. 28 shows the ground condition before treatment. N_{SPT} was between 0 and 2 with CPT $q_c < 0.3$MPa. The ground water table was about 2m below surface. The height of the embankment varied from 8m to 10.5m. Large diameter up to 3.5m dynamic replacement columns were installed using 200 - 400mm size rock pieces at a grid of 5m giving an area replacement ratio of 28%. Plate bearing tests on single columns were carried out registering 5mm settlement at 200kPa and 7mm settlement at 300kPa. Full embankment height was reached in less than 3 months. A 2m-surcharge was applied for 8 weeks. Measured settlement during embankment construction was between 15cm and 30cm. After 6 weeks of surcharging, the rate of settlement was less than 0.1mm per day.

Figure 28. Site condition before treatment (top); 300 ton.m DR rig (middle) and 3.5m diameter DR rock column (bottom) at Senai.

The estimated dynamic replacement treatment area for railway construction would probably exceeded 600,000m^2 by the end of 2006.

Yee & Varaksin (2006) and Ong *et al.* (2006) reported the use of dynamic replacement columns to support large cylindrical tanks on marginal ground. Tanks up to 76m diameter have been successfully supported on DR columns. The specific foundation treatment for tanks follows a specific earthworks procedure in combination with dynamic replacement columns for foundation support and settlement reduction. During the period of 1997 – 2000, many petrochemical projects were building tank farms at Kerteh, Gebeng and Kuantan such as Optimal, Kerteh Centralised Tankage Facilities, Malaysian Acetyl, etc. and these tanks are founded on treated ground resting on compacted sand pad rather than on RC slab on piles.

6.3 Controlled Modulus Columns

Controlled modulus columns (CMC) are semi-rigid cement grout columns. Unlike a rigid RC pile, there is a sharing of load between the CMC and the surrounding soil facilitated by the load distribution layer (sand) as shown in Fig. 29. Typical diameter of CMC ranges from 15cm to 50cm with a design load capacity ranging from 8 – 50 tons. These columns are typically 10 – 20m length with larger diameter columns installed to 30m depth. These columns are designed to achieve a pre-determined stiffness compared with that of the surrounding soil.

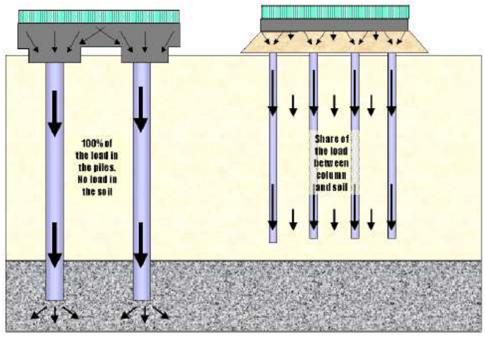

Figure 29. Concept of CMC columns compared with RC piles

The technique of CMC was developed as an extension of the conventional soil cement mixing technique. The conventional technique uses a rotary tool which is “dough mixer” or “egg beater” shaped to form columns. The formation of the column is done by rotating the rotary tool to mix the soil and the cement grout. Hence, its application in sensitive soft clay is limited due to the remoulding effect of the mechanical mixing process on the shear strength of the in-situ soil. The alternative solution is to form columns by displacement method.

The installation of CMC by displacement involves an augering process which is vibration-free and quiet. It uses a special auger powered by an equipment of large torque capacity and high static down thrust. During augering, it displaces the soil laterally and hence, compact the surrounding soil to form displacement column, thus minimize amount of spoil. Once the required depth of installation is reached, cement grouting of the column takes place under controlled pressure (usually less than 5 bars) to ensure a perfect soil-cement grout contact. During the extraction of the auger, continuous grouting under controlled pressure takes place. The result is a cement column shaft that is effectively bonded to the surrounding soil. During installation, the torque and the rate of penetration of the auger is closely monitored. Similarly, during the formation of the column, the rate of the auger extraction is controlled with respect to the cement grout mixture flow rate. A measuring gauge is used to maintain the supply of cement grout mixture which will also be used to indicate the column diameter with respect to depth. Fig. 30 shows the CMC installation process.

Ting *et al.* (2004) reported the application of displacement cement columns to support the raised embankment of a dam in Sarawak. The installation of the columns was carried out during the full operation of the dam where progressive fissuring due to overstressing of the in-situ material may prove to be fatal. The displacement columns scheme was designed to limit the volumetric strain to less than 3%. The grouting pressure was limited to the yield strength of the in-situ material of 1 bar. The ratio of the stiffness of columns to the in-situ material was kept at 4:1 to limit the differential settlement of 1:500 between the columns. The maximum allowable shrinkage of grouted columns was limited to 0.5% to alleviate piping problems. Fig.

31 shows the cross-section of the raised embankment and the trial column.

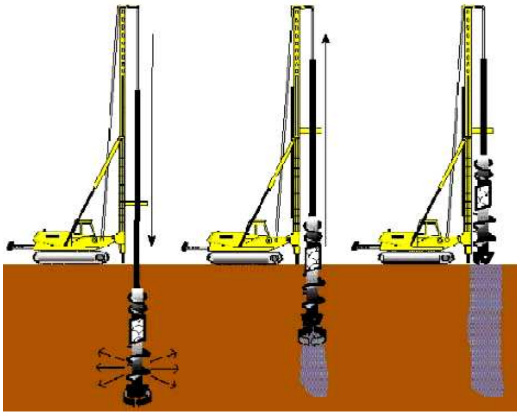

Figure 30. Installation of CMC by displacement

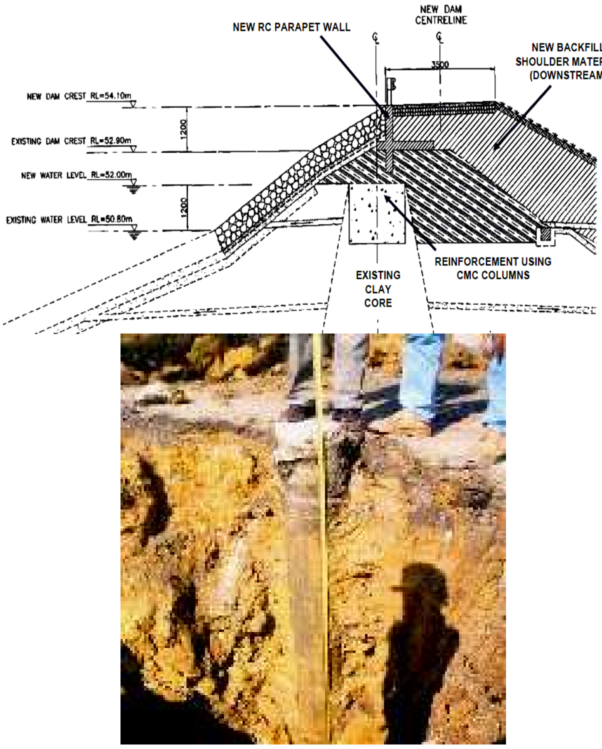

Figure 31. Cross-section of raised embankment (top) and trial column (bottom) at Sebubut Dam.

6.4 Suitable Types of Soil for Reinforcement Treatment

Soft cohesive soil is improved by consolidation (e.g. vertical drainage with surcharge fill) and it takes time to achieve the required degree of consolidation. Alternatively, ground reinforce-ment can be applied in soft soil to increase bearing support and provide stability in shorter time. Also, these reinforcement columns can reduce post construction settlement by a factor of 2 to 4. However, typical treatment depth is between 10 – 15m and in exceptional case it goes deeper than 20m.

6.5 Design Issues

The design of reinforcement columns requires the determination of (1) column diameter; (2) spacing; (3) friction angle of backfill material (or stiffness of columns); (4) shear strength of the native soil (or stiffness of surrounding soil); (5) stress ratio between reinforcement columns and surrounding soil (area replacement ratio); and (6) strain compatibility between columns and surrounding soil.

During the installation of stone columns, due to vibration the stones are forced radially into the surrounding soil forming a stone column that is tightly interlocked with the soil. The installation of stone columns transforms the ground into a composite mass of cylindrical columns of stones with intervening native soil, providing a lower compressibility and higher shear strength than those of the native soil alone. Since the stiffness of the stone column is substantially higher than that of the surrounding soil, a larger portion of the applied load is transferred to the column (defined by the stress concentration factor), thus improving the load-carrying capacity of the treated ground and reducing its settlement. However, the column material is cohesionless stone. Its stiffness depends upon the lateral support given by the soil surrounding it. If that support is inadequate, the column bulges and ground deformation increases.

The load carrying capacity of a stone column is a function of the column diameter, angle of internal friction of the stone and shear strength of the in-situ soil (Fig. 32), among other parameters. The column diameter is determined by the method of installation, the stone size and the strength of the in-situ soil. Typically, it ranges from 0.8m to 1m. Wet method installation tends to give larger diameter columns, generally about 1m diameter due to the water flushing of spoil. Dry method installation is usually about 0.8m diameter. The angle of internal friction (ϕ_s) of the stone column depends on the size and shape of the stone used, the installation method and the infiltration of the in-situ soil between stone particles. For top feed wet method, typical stones used are 45 – 70mm in size, crushed stone with ϕ_s about 42^o – 45^o. For bottom feed dry method, the stones used are smaller and less angular to avoid blockage of the feeder tubes. Generally, rounded stones of 15 – 35mm in size with ϕ_s of 38^o – 42^o are used with bottom feed method.

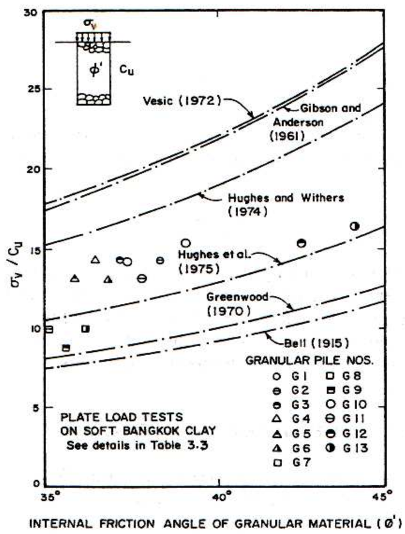

Figure 32. Bearing capacity of a stone column (Bergado & Lam, 1987)

The bearing capacity of a foundation founded on stone columns is equal to the capacity of a single column multiplied by the number of columns (FHWA, 1983). The ultimate bearing capacity of a single column is expressed by:

$$\sigma_{ult} = N_{sc}.c_u \tag{3}$$

where σ_{ult} is the ultimate column capacity; N_{sc} is the bearing capacity factor for stone column ($18 \le N_{sc} \le 22$) and c_u is the undrained shear strength of the soil. Jie Han (2010) suggested $\sigma_{ult} = 20c_u$ and $c_u > 15$ kN/m^2 for stone column applications.

Pseudo-elastic and elasto-plastic theories are used to calculate the settlement of the composite column-soil mass using the unit cell concept. A unit cell represents the area tributary to one stone column. Fig. 33 is a compilation of settlement ratio curves (Greenwood & Kirsch, 1984). These curves relate the settlement improvement ratio (settlement of soil without stone columns to that with stone columns) to the area ratio (area of unit cell divided by area of stone column).

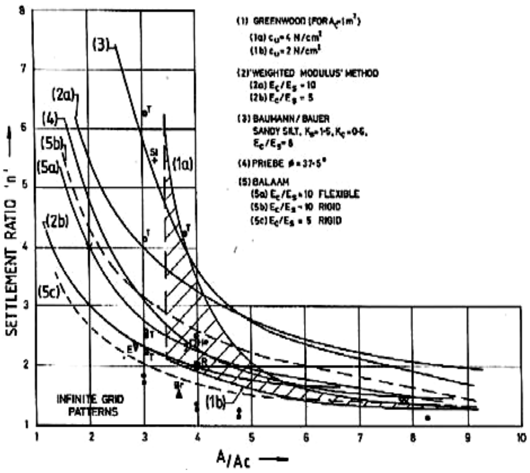

Figure 33. Settlement ratio curves with reinforcement granular columns (Greenwood & Kirsch, 1983)

For slope stability analysis, composite shear strength is used along the sliding surface. The composite strength parameters are related to the shear strength of the soil, the friction angle of the column, the stress ratio and the area replacement ratio.

For dynamic replacement columns, the methods of analysis for bearing capacity and settlement calculation for stone columns are equally applicable considering that both are non-rigid granular columns. Yee & Varaksin (2007) reported the use of in-situ pressuremeter test results (limit pressure and pressuremeter modulus) to calculate bearing capacity and settlement for large tanks founded on improved ground by dynamic replacement columns using the direct design approach.

However, the method of analysis for semi-rigid CMC is different from non-rigid granular columns. The ratio of stiffness of CMC to the surrounding soil is much higher. The stiffness modulus of CMC is typically 5,000 MPa as compared with 30 – 80 MPa for non-rigid granular columns. Details of the design methodology for CMC is given in Plomteux & Liausu (2007) and Plomteux & Lacazedieu (2007).

CMC is also used for anti-liquefaction treatment under seismic condition. The design of CMC in this case incorporates the effect of volumetric strain on the surrounding soil and the increase shear resistance of the composite CMC-soil mass to resist lateral displacement and shear stresses induced during a seismic event. During installation, the displacement auger displaces the soil laterally without extraction of spoil and hence, increases the density of the soil which reduces the susceptibility of liquefaction. Menard (1975) suggested that a volumetric strain of 4% in sand will result in an immediate compaction which increases the limit pressure by a factor of 2 and hence, it increases the bearing capacity accordingly (Table 2).

Table 2. Influence of soil displacement against increased in bearing capacity (Menard, 1975)

Volumetric Strain	**Improvement Factor**		
	Sand	**Silt**	**Clay**
1%	1.3	1.2	1.1
2%	1.5	1.4	1.2
4%	2.0	1.6	1.3

6.6 Performance Evaluation

The performance of granular columns can be evaluated by in-situ tests such as pressuremeter tests and plate bearing tests. Cone penetration tests are carried out in sand columns but not stone columns or rock columns. From the pressuremeter tests, one can obtained the stiffness of the column as well as the surrounding soil when the tests are carried out inside the columns and in-between the columns respectively. The pressuremeter test is in fact, a direct test to simulate the column bulging behaviour in stone columns. The stiffness values obtained can be used in re-analysing the settlement prediction following the direct design approach. Instrumentation and monitoring for vertical settlement and lateral deformation are usually carried out during construction.

6.7 Choice of Reinforcement Methods and Selection Criteria

The determinative factor in the choice of reinforcement method is the depth of columns and the amount of tolerable post construction settlement. Beside, environmental constraint needs to be considered. Dynamic replacement produces the greatest vibration followed by vibro stone columns. CMC is vibration-free and is suitable for construction very close to sensitive structure or in urban areas with environmental constraints. For dynamic replacement, a cut-off trench of 1.5m deep is usually constructed to intercept the surface vibratory wave which causes surface vibration. With trenching, it is possible to reduce the safe distance to half or reduce the vibration by 50%.

Dynamic replacement columns are typically 5 – 7m length with deeper columns requiring higher compaction energy and pre-excavation of column craters (“prints”). Typical stone columns are 10 – 15m with deeper columns extending to 25m but seldom used while typical CMC columns are 10 – 20m.

Dynamic replacement and vibro stone columns are considered non-rigid “flexible” columns and the settlement improvement ratio is typically 2 to 4. The settlements after improvement are typically larger than those treated with CMC since CMC columns are more “rigid” than granular columns. When CMC columns are founded on firm stratum, the settlements are usually much lesser compared with granular columns. Typical settlement is in the range of 10 – 20cm in most cases.

Dynamic replacement is cost-effective for large treatment areas (> 10,000m^2) with its high productivity but only if the surface vibration is not an issue of concern. The choice of backfill materials is flexible with option of using sand, aggregate, rock pieces or even construction debris (bricks, broken concrete blocks, etc.) or a mixture of hard, durable and inert materials whichever is cheaper. Vibro stone column technique is a cost-effective solution when there is a readily cheap supply of stones and a source of water within close distance. Otherwise, dry method of installation has to be carried out which has a lower productivity compare to the wet method. Stone columns are generally not used when disposal of sludge is a problem (for wet method); stone stockpile area is limited or sensitive structures or utilities within close distance. CMC columns is fast gaining a reputation as an environmental friendly solution. The installation is vibration-free, quiet and minimum spoil to dispose. It has a smaller post construction settlement and most suited for urban works where cement grout can be readily obtained. Behaviour of CMC is somewhat between stone columns and RC piles.

7. SUSTAINABLE DEVELOPMENT AND LOW CARBON ECONOMY

Sustainable development is defined as “...development that meets the needs of the present without compromising the needs of the future...”. The need for sustainability is described in Yee

& Ooi (2007). To sustain means to "hold up" and it is primarily the role of geotechnical engineers to improve foundation designs and construction processes that hold up the structures to be built on it with less materials, less energy usage and generate less CO_2. It may not be able to achieve zero-energy design but the move towards a low carbon economy of recycling and using alternative low-carbon construction processes often proved to be more profitable for the contractor as well as better value for the client. A number of case studies are presented below. For the calculation of carbon footprint, the CO_2 emision for materials is based on manufacturing process while for operations, it is based on CO_2 emission during works.

The first case study involved the construction of a warehouse with an option of using deep piled foundation or shallow foundation on treated ground using dynamic compaction. Table 3 shows a CO_2 emission audit exercise carried out for the construction of a warehouse (Liausu, 2007). The warehouse is 20,000m^2 with a working load of 5 ton/m^2. The ground conditions consisted of a non-engineered unsaturated fill down to 6m depth. There are two possible solutions; (i) deep foundation with 25cm thick RC slab on piles; and (ii) shallow foundation with loose fill improved by dynamic compaction with 15cm thick RC slab and a 30cm thick compacted granular layer acting as a load transfer layer between the RC slab and the improved ground. Using the shallow foundation solution helped to reduce the overall carbon footprint by approximately 500 tons CO_2 representing an offset for CO_2 emission of about 110 persons for a year based on a per capital of 4.5 tons CO_2 (UNDP, 2007)

Table 3 CO_2 emission audit on a warehouse construction project

Deep foundation - 1 pile per 20m^2 with 50cm diameter pile and 7.5m length - Concrete for 25cm thick slab		Shallow foundation - 1 DC rig and 1 shovel - Induced settlement of 50cm - 30cm thick sand blanket - Concrete for 15cm thick slab	
Piling operation	0.49 kg	Crane fuel	0.81 kg
Concrete for piles	15.0 kg	Shovel fuel	0.48 kg
Additional concrete (0.1m^3/m^2) for slab compared with shallow foundation	20.0 kg	Sling / pounder	0.35 kg
		Fill for settlement	4.50 kg
		Sand blanket	4.90 kg
Kg eq. CO_2 per m^2	35.5 kg	Kg eq. CO_2 per m^2	11.0 kg
Saving of CO_2 for shallow foundation per m^2			24.5 kg
For the warehouse of 20,000m^2, total saving CO_2			490 tons

The second case study involved the construction of a storage terminal with two options of ground improvement; either using vertical drains with 4m of fill surcharge or vacuum consolidation with 0.8 bars of vacuum depressurization to treat the underlying soft cohesive soil. Table 4 shows a CO_2 emission audit exercise carried out for the construction of the storage terminal (Liausu, 2007). The storage terminal is 100,000m^2. The ground conditions consisted of soft marine clay down to 20m depth. With vertical drains and fill surcharge, the closest source of fill material for surcharge was 10km away from the site. Any surplus fill material at the end of the consolidation period was required to be taken offsite. Alternatively, vacuum consolidation provided a possible mean to do without importing the fill material for surcharging. A design vacuum pressure of 0.8 bars was used to replace the 4m fill surcharge. For a large treatment area, this leads to a substantial saving in the volume of imported fill materials, reduced cost of transportation and earthmoving operation. Using the vacuum consolidation solution helped to reduce the overall carbon footprint by approximately 1,100 tons CO_2 representing an offset for CO_2 emission of about 245 persons for a year. In this case, vacuum consolidation was more sustainable than vertical drains with 4m fill surcharge and also proved to be more economical.

Table 4. CO_2 emission audit on a container terminal project

Vertical drains and fill surcharge - 4m surcharge fill - Source of fill is 10km away - Vertical drain of 20m length at 1.5m grid		Vacuum consolidation - 1.5mm thick HDPE membrane - Sealing trenches - Vacuum pumping (12kW per 2,500m^2) - Vertical drain of 20m length at 1.5m grid; no surcharge fill	
Loading of fill at source	1.00 kg	HDPE membrane	0.75 kg
Transportation	5.60 kg	Horizontal drains	0.10 kg
Place surcharge fill	0.70 kg	Trenches	0.16 kg
Removal of surcharge fill	0.70 kg	Vacuum pumping	1.65 kg
Transport offsite	5.60 kg		
Kg eq. CO_2 per m^2	13.6 kg	Kg eq. CO_2 per m^2	2.6 kg
Saving of CO_2 for vacuum consolidation per m^2			11.0 kg
For the container terminal of 100,000m^2, total saving CO_2			1,100 tons

Note: The vertical drain design and the induced settlement are the same for both options and hence, the carbon footprint computations are not included above.

Table 5. CO2 emission audit on a road embankment project

Removal and replacement - 550,000m^3 of unsuitable materials to be excavated and removed. - Transportation of 650,000m^3 of suitable fill materials - 550,000m^3 of compacted fill volume. - Dewatering process		Dynamic replacement and vertical drains - 1 DR rig, 1 PVD rig and 1 shovel - Induced settlement of 35cm - 147,000m^3 of DR backfill - Vertical drain of 6.5m length at 1.35m grid; no surcharge fill	
Dewatering, excavate unsuitable materials and transport offsite	18.2 kg	DR rig fuel	0.95 kg
		PVD rig fuel	0.50 kg
		Shovel fuel	0.45 kg
Loading of suitable material at source and transport to site	12.5 kg	Sling / pounder	0.07 kg
		Vertical drains	0.34 kg
Dewatering, bunding and temp. platform, geotextile, placing and layer compaction	6.71 kg	Fill for settlement and DR columns	3.04 kg
Kg eq. CO_2 per m^2	37.4 kg	Kg eq. CO_2 per m^2	5.35 kg
Saving of CO_2 for DR and PVD per m^2			32.0 kg
For the treatment area of 102,000m^2, total saving CO_2			3,264 tons

Note: The sand blanket is considered as part of embankment fill material and hence, the carbon footprint computation is not included above.

The third case study involved the construction of an average 10m high road embankment founded on soft peaty clay of 5m thick below the surface. It was envisaged to remove and replace the soft peaty clay. The ground water was at the surface. The distance of the dumping site for the excavated material and the source of suitable fill was about 20km. Due to construction problems associated with high ground water table, high cost of imported fill materials and time constraint, alternative solution of ground improvement was carried out on an area of 100,000m^2. Given a 12-month construction period, the consolidation of the soft peaty clay has to be accelerated. The required drainage was provided by the dynamic "sand-and-stone" replacement (DR) columns and vertical drains which were installed in-between the DR columns. In addition, the DR columns also served as a bearing support to the embankment so that the required rate of filling of embankment can be achieved. Without DR columns, the rate of filling as well as the height of embankment may be limited. Table 5 shows a CO_2 emission audit exercise carried out for the foundation works. Using this combined solution of DR and vertical drains had allowed the construction to meet the time constraint and helped to reduce the overall carbon footprint by more than 3,000 tons CO_2 representing an offset for CO_2 emission of about 700 persons for a year.

In the above case study, some of the backfilling materials for the DR columns were construction debris (broken bricks and concrete pieces) obtained from a nearby demolition project uderlying the potential use of recycled materials.

An attempt is carried out to quantify the CO_2 emission of some commonly used ground improvement techniques. Table 6 shows the CO_2 emission based on installation works only excluding CO_2 emission associated with material production and earth-moving operations. For example, CO_2 emission is calculated for installation of PVD material in a vertical drain project while in vacuum consolidation, CO_2 emission is calculated for installation of PVD, horizontal drains and HDPE membrane. Placing of surcharge and perimeter trenching (under earthmoving) are not included. Hence, it is not a complete and a detailed calculation of total carbon footprint which is needed for each project as it is site specific and subject to the initial ground conditions and the performance specifications which determine the extent of ground improvement required. Sources of materials, travelling distance and site accessibility are also to be considered. Generally, newer and more advanced equipment are designed for lower CO_2 emission. Hence, the technology employed and the age of equipment used affect its working capacity and fuel efficiency which influence the CO_2 emission with respect to productivity. So, different projects will have different carbon footprint values.

Table 6. Carbon footprint for some ground improvement techniques based on installation process (excludes material production and earthmoving operation)

Ground improvement methods	**Treatment depth**	**CO_2 emission associated with installation works**
Vertical drains @ 1.5m grid	20m	1.62 kg/m^2
Vacuum consolidation	20m	1.71 kg/m^2
Dynamic compaction	10m	2.53 kg/m^2
Vibro compaction	10m	3.09 kg/m^2
Dynamic replacement @ 5m grid	5m	2.16 kg/m^2
Vibro replacement @ 2.5m grid	10m	5.76 kg/m^2
Controlled modulus columns @ 2m grid	10m	4.50 kg/m^2

8 CONCLUSIONS

Ground improvement has been introduced in Malaysia since 1978 and the experience has come of age. It presents the engineer a solution to marginal ground - the engineer "forces" the ground to meet the project's requirements by altering its natural state, instead having to change his design to meet the ground's limitations.

At the same time, it has also increased awareness of its limitations as each technique has its own merits, limitations and economies. Ground improvement requires specialized and intensive engineering input. It requires a more detailed and elaborate site investigation as well as a detailed performance monitoring program. Estimates of bearing capacity and settlements still require post-treatment in-situ tests such as pressuremeter tests, cone penetration tests, plate bearing tests, etc. Instrumentation of soil response still plays an important role in the success of a ground improvement project.

In addition to the benefits of rehabilitation of marginal ground for development, ground improvement is also a sustainable construction method. Bandar Sunway is a good example.The future of ground improvement is evidence that it is firmly founded on a path of continuing development with improved equipment, refined methods of analysis, improvement in the field and laboratory testing of soils and objective performance evaluation. All these could only increase the technical and economic advantages of ground improvement.

The objectives of this paper have been to review the opportunities and constraints of each ground improvement techniques; to provide an awareness of on-site adaptation of specific design and construction process to suit the prevailing ground conditions. This paper also calls for sustainable development using low-carbon technology and to introduce carbon footprint accounting practice. The need to provide CO_2 emission audit during technical and commercial evaluation of a project and to present them in a way that can be easily communicated to the client and allowing such choices to be made early on in a project cannot be over-emphasized. It is an important responsibility of the engineers. Going green is no longer an option.

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10. REFERENCES

Ali, F., Yee, K. & Varaksin, S. 1997. Treatment of Highly Compressible Soils. *Proceedings of the Conference on Recent Advances in Soft Soil Engineering*, Kuching, Sarawak; 46 – 65

Awang, A., Yee, K. & Marzuki, M. 1987. Vertical Drains and Preloading. *Proceedings of the Geotechnical Seminar on Practical Applications of Ground Improvement Techniques, Kuala Lumpur.*

Barron, R. A. 1948. Consolidation of Fine Grained Soils by Drain Walls. *Trans. ASCE*, Vol. 118; 718 – 754

Bergado, D. T. & Lam, F. L. 1987. Full Scale Load Test of Granular Piles with Different Densities and Different Proportions of Gravel and Sand in Soft Bangkok Clay, *Soil & Foundations Journal*, Vol. 27 No. 1; 86 – 93.

Choa, V. & Bawajee, R. 2002. The Changi Experience. *Proceedings of the IES Conference on Case Studies in Geotechnical Engineering*, NTU, Singapore; 63 – 89.

Cognon, J. M. 1991. Vacuum Consolidation. *Rev. French Geotechnique,* No. 57(3); 37 – 47.

Cognon, J.M., Juran, I. & Thevanayagam, S. 1994. Vacuum Consolidation Technology – Principles and Field Experience. *Vertical and Horizontal Deformations of Foundations and Embankments*, Geotechnical Special Publication No.40, ASCE;.1237 – 1248

D'Appolonia, E. 1970. Dynamic Loading. *Journal of Soil Mechanics and Foundation Engineering ASCE* Vol. 96 No. SM1

EPU. 2003. Mid-Term Review of the 8th Malaysia Plan 2001 - 2005. *Economic Planning Unit, Prime Minister's Department, Malaysia.*

FHWA. 1983. Design and Construction of Stone Columns. US Department of Transport, Federal Highway Administration

Glover, J. C. 1982. Sand Compaction and Stone Columns by the Vibroflotation Process. *Symposium on Recent Developments in Ground Improvement Techniques,* Bangkok; 3 – 15

Greenwood, D. A. & Kirsch, K. 1983. Specialist Ground Treatment by Vibratory and Dynamic Methods. *Proceedings of the International Conference on Advances in Piling and Ground Treatment for Foundations*. London; 17 – 45

Hansbo, S. 1979. Consolidation of Clay by Band-shaped Prefabricated Drains. *Ground Engineering*, Vol. 12, No. 5

Han, J. 2010. Short Course on Ground Improvement, 21 – 22 July 2010 organised by AGSSEA.

Kjellman, W. 1952. Consolidation of Clay Soil by Means of Atmospheric Pressure. *Proceedings of Conference on Soil Stabilization, Massachusetts Institute of Technology, Cambridge;* 258 – 363.

Lee, S. L., Yong, K. Y., Tham, K. W., Singh, J. & Chen, W. P. 1989. Treatment of Ex-Mining Land by Fibredrains, Surcharge and High Energy Impact. Proceedings of the Symposium on the Application of Geosynthetic and Geofibre in Southeast Asia, Petaling Jaya; 5-18 – 5-22

Liausu, P. 2007. Private Communications.

Massarch, K. R. & Heppel, G. 1991. Deep Vibratory Compaction Using the Muller Resonance Compaction System. Report 91:2 for Muller Geosystems.

Meigh, A. C. 1987. Cone Penetration Testing – Methods and Interpretation. CIRIA, 141 p

Menard 1975. The Menard Pressuremeter – Interpretation and Application of Pressuremeter Test Results. Sols Soils No. 26, 43 p.

Menard, L. & Broise, Y. 1975. Theoretical and Practical Aspects of Dynamic Consolidation. *Geotechnique,* Vol. 25 No. 1

Mitchell, J. K. 1981. Soil Improvement State-of-the-Art Report. 10th ICSMFE, Stockholm, Vol. 4; 509 – 565.

Ong, R., Yee, K. & Wong, L. T. 2006. Dynamic Compaction and Dynamic Replacement for Large Oil Tank Foundations. *Proceedings of the 16th Southeast Asian Geotechnical Conference, Kuala Lumpur*

Ooi, T. A. 1997. Some Aspects of Ground Improvement Works for the New Kuching Deep Water Port at Kampung Senari, Sarawak. *Proceedings of the Conference on Recent Advancesin Soft Soil Engineering,* Kuching, Sarawak; 46 – 65.

Ooi, T. A. & Yee, K. 1997. Some Recent Developments of Ground Improvement Technology in Southeast Asia. *Proceedings of the 30th Year Anniversary Symposium of the SEAGS on Deep Foundations, Excavations, Ground Improvements and Tunnelling,* Bangkok; 1-81 to 1-106.

Ooi, T. A. 2004. Earthwork Practice in Malaysia. Special Lecture *Proceedings of the Malaysian Geotechnical Conference*, Kuala Lumpur; 45 – 58

Ooi, T. A. 2007. Private Communications.

Ooi, T. A. & Ooi, H. M. 2009. Geotechnical Failures / Issues, Dispute Resolution and Mitigation. *19th Professor Chin Fung Kee Memorial Lecture,* 7th Nov. 2009, Kuala Lumpur.

Plomteux, C. & Lazacedieu, M. 2007. Embankment Construction on Extremely Soft Soils using Controlled Modulus Columns for Highway 2000 Project in Jamaica. *Proceedings of the 16th Southeast Asian Geotechnical Conference,* Kuala Lumpur.

Plomteux, C. & Liausu, P. 2007. Construction of Flat Glass Float Factory on Loss Deposits using Controlled Modulus Columns in Seismic Conditions. *Proceedings of the 16th Southeast Asian Geotechnical Conference,* Kuala Lumpur.

Risseeuw, P., Volders, R. & Teunissen, H. 1986. Soil Improvement by Preloading and Vertical Drains, Port Kelang Power Station. Malaysia. *Proceedings of the 3rd International Conference on Geotextiles,* Vienna; 599 – 604

Tan, Y. C. & Liew, S. S. 2000. Observational Method to Prevent Failure of Embankment Treated with Vacuum Preloading. *Seminar on Failures Related to Geotechnical* Works. Institution of Engineers, Malaysia, Kuala Lumpur.

in Ground Improvement Techniques, Balkema.

Ting, W.H. 1982. Some Examples of Ground Improvement in Malaysia. *Proceedings AIT Symposium on Recent Development in Ground Improvement Techniques,* Balkema.

Ting, W. H., Hassan, Z. & Daulah, I. 1982. Soil Improvement for Foundation of a Housing Project by Dynamic Consolidation Method. *Internal Report* (Unpublished).

Ting, W. H. 1985. Foundation in Limestone Areas of Malaysia. *Proceedings of the 8th Southeast Asian Geotechnical Conference*, Kuala Lumpur Vol. 2; 125 – 136

Ting, W. H., Nithiaraj, R. & Ng, S. J. 1992. The Performance of Treated Mining Pond Slime. *Symposium on Prediction Versus Performance in Geotechnical Engineering*. AIT/SEAGS

Ting W. H., Varaksin, S., Tan K. P. & Spaulding, C. 1995. Vacuum Consolidation and Dynamic Replacement Columns for Embankment Foundation on Mining Slimes. *Proceedings of the 10th Asian Regional Conference on Soil Mechanics and Foundation Engineering, Beijing.* Vol. 1; 453 – 456.

Ting, W.H. 1998. The Application of Ground Treatment in Malaysia. Short course on Geotechnical Engineering. *The Institution of Engineers, Malaysia*, Kuala Lumpur.

Ting, W. H., Yee, K., Tai, L. Y. & Ong, D. E. L. 2004. Support of Raised Embankment of a Small Dam by Cement Columns. *Proceedings of the Malaysian Geotechnical Conference*, Kuala Lumpur; 467 – 474

Toh, C. T., Ooi, T. A. & Ting, W. H. 1985. Behaviour of a Raft Foundation on Improved Soil and Rock. *Proceedings of the 8th SEAGC,* Kuala Lumpur; 3-64 to 3-80.

UNDP 2007. United Nations Development Programme. Human Development Report 2007/2008. *Palgrave Macmillan, New York*, 2007. 399 p.

Varaksin, S. & Yee, K. 2007. Challenges in Ground Improvement Techniques for Extreme Conditions: Concept and Performance. *Proceedings of the 16th Southeast Asian Geotechnical Conference*, Kuala Lumpur.

Venables, R. 2001. Delivery Sustainable Development. *ICE Brunel Lecture 2001*, Institution of Civil Engineers, London.

Yee, K. 1990. The Use of Geotextiles in Reclamation of Ex-Mining Ponds for Housing Development and Their Purposes. *Proceedings of the Post Cambridge Seminar on Reinforced Embankments and Slopes: Theory & Practice,* Singapore.

Yee, K. 2000. Consolidation of Soft Clay by Vertical Drainage Method. *Proceedings of the International Conference on Soft Clay Engineering*, Institute of Engineers, Malaysia and University of Technology, Malaysia, Kuala Lumpur.

Yee, K. & Ooi, T.A. 2003. Opportunities and Constraints for Ground Improvement Techniques in High Speed Railway Embankment Construction – A Malaysian Perspective. *Proceedings of Railtech 2003 "Railway Technology in the New Millennium – The Malaysian Transition"*, Kuala Lumpur

Yee, K. & Ooi, T. A. 2009. Green Technology for Sustainable Foundation Treatment of a High Embankment. *Proceedings of the International Symposium on Ground Improvement Technologies and Case Histories,* Singapore.

Yee, K., Ooi, T. A. & Ting, W. H. 2004. Vacuum Consolidation Technology for Soft Soils Treatment – Principles and Field Experiences. *Proceedings of the Malaysian Geotechnical Conference*, Kuala Lumpur; 561 – 570

Yee, K. & Ting, W. H. 2004. Ground Improvement to Accelerate Construction of a High Embankment. *Proceedings of the Malaysian Geotechnical Conference,* Kuala Lumpur; 529 – 538

Yee, K. & Varaksin, S. 2007. Settlement Performance of Large Steel Tanks on Soft Soils Improved by Large Diameter Granular Columns. *Proceedings of the 16th Southeast Asian Geotechnical Conference,* Kuala Lumpur.

Yeow, T. S., Wong, B. C. & Ooi, T. A. 1993. Some Aspects of Geotechnical Consideration in the Housing Development on Ex-mining Land" *Proceedings Joint IEM-GSM Forum on Urban Geology and Geotechnical Engineering in Construction*, Petaling Jaya; p. 4-1 to 4-18.