Ask Vincent Chu
(Common FAQ on Practical Civil Engineering Works)

Vincent T. H. CHU

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Preface

This is my third book since my first one in 2006. Following positive and encouraging response since the publication of "200 Questions and Answers on Practical Civil Engineering Works" and "Civil Engineering Practical Notes A-Z", it provides great incentive for me to further write and discuss civil engineering practice to share my knowledge with fellow engineers around the world.

Ever since the establishment of the free email service "Ask Vincent Chu" in 2008, a huge surge of email were received from time to time regarding civil engineering queries raised by engineers around the globe. It is my interest to publish some of these engineering queries in this book and hence the title of this book is called "Ask Vincent Chu". Moreover, in this book I intend to write more on geotechnical aspects of civil engineering when compared with my previous two publications.

Should you have any comments on the book, please feel free to send to my email *askvincentchu @yahoo.com.hk* and discuss.

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Chapter 1. Bridge Works

1. What is the purpose of dowel bar in elastomeric bearing?

Elastomeric bearing is normally classified into two types: fixed and free. For fixed types, the bridge deck is permitted only to rotate and the horizontal movements of the deck are restrained. On the other hand, for free types the deck can move horizontally and rotate. To achieve fixity, dowels are adopted to pass from bridge deck to abutment. Alternatively, in case there is limitation in space, holes are formed in the elastomeric bearings where anchor dowels are inserted through these holes. It is intended to prevent the "walking" of the bearing during its operation.

2. What is the importance of shear stiffness in the design of elastomeric bearing?

For elastomeric bearing, the shear stiffness is an important parameter for design because it influences the force transfer between the bridge and its piers. In essence, elastomers are flexible under shear deformation but it is relatively stiff in compression. However, elastomeric bearings should not be used in tension.

Elastomeric bearing should be designed in serviceability limit state only. The cross sectional area is normally determined by the compressive stress limit under serviceability limit state. The shape factor, i.e. plan area of the laminar layer divided by area of perimeter free to bulge, affects the relation between shear stress and the compressive load. In essence, higher capacity of bearings could be obtained with higher shape factor.

The long side of the bearing is usually oriented parallel to the principle axis of rotation because it facilitates rotational movement. The thickness of bearings is limited and controlled by shear strain requirements. In essence, the shear strain should be less than a certain limit to avoid the occurrence of rolling over at the edges and delamination due to fatigue. Hence, it follows that higher rotations and translations require thicker bearing. On the other hand, the vertical stiffness of bearings is obtained by inserting sufficient number of steel plates. In addition, checks should be made on combined compression and rotation to guard against the possible occurrence of uplifting of corners of bearings under certain load combinations.

3. What are the limitations of grillage analysis?

In designing the number of cells for concrete box girder bridges, in case the depth of a box girder bridge exceeds 1/6 or 1/5 of the bridge width, then it is recommended to be designed as a single cell box girder bridge. However, if the bridge depth is smaller than 1/6 of the bridge width, then a twin-cell or multiple cell is a better. However, one should note that even for wider bridges with small depths, the number of cells should be minimized because there is not much improvement in transverse load distribution when the number of cells of box girder is increased to three or more.

For structural analysis of bridges, grillage analysis, which involves the structure to be modeled as a series of longitudinal and transverse elements which are interconnected at nodes, is normally adopted.

Grillage analysis suffers from the following shortcomings based on E. C. Hambly:

(i) For coarse mesh, torques may not be identical in orthogonal directions. Similarly, twists may differ in orthogonal directions.
(ii) Moment in any beams is mainly proportional to its curvature only. However, moment in an element depends on the curvatures in the beam's direction and its orthogonal direction.

Grillage analysis cannot be used to determine the effect of distortion and warping. Moreover, the effect of shear lag can hardly be assessed by using grillage analysis. By using fine mesh of elements, local effects can be determined with a grillage. Alternatively, the local effects can be assessed separately and put in the results of grillage analysis.

4. For the loading pattern to obtain maximum positive moment in a span of a continuous beam, why should alternative spans on each side of the span be loaded?

To acquire a maximum sagging moment in a span of a continuous beam, the general rule is to load the span under consideration and alternative spans on each side of the span. To account for this rule, let's consider the following example. For instance, loads are applied to the mid-span of a multiple-span continuous beam. It is noticed that this loads induce positive moments near mid-span in all even spans. Therefore, if all even spans are loaded simultaneously, this will result in the increase of positive moments in all other loaded spans.

Similarly, to obtain maximum negative moment at a support, load adjacent spans of the support and then alternative spans on each side.

5. What are the main potential benefits in using the bridge form of precast prestressed beams supporting in-situ concrete top slab?

The potential benefits of using the bridge form of precast prestressed beams supporting in-situ concrete top slab are:

(i) For bridges built on top of rivers and carriageway, this bridge form provides the working platform by the precast beams so that erection of falsework is not required.
(ii) This bridge form generally does not require any transverse beams or diaphragms (except at the location of bridge supports), leading to reduction of construction time and cost.
(iii) It creates the potential for simultaneous construction with several spans.

6. What are the potential advantages in using lightweight aggregates in bridges?

The advantages in using in using lightweight aggregates in bridges:

(i) Owing to reduced dead load by using lightweight aggregates, there are savings in structural material such as the cost of foundation and falsework.

(ii) It brings about environmental benefits when industrial waste products are used to manufacture lightweight aggregates.

(iii) It enhances higher durability by having lower coefficient of thermal expansion which reduces the thermal movement. Moreover, it has lower permeability and higher resistance to freeze-thaw cycles when compared with normal aggregates.

7. How does flatter affect the stability of long bridges?

Flutter is a potentially destructive vibration and it is self-feeding in nature. The aerodynamic forces on a bridge, which is in nearly same natural mode of vibration of the bridge, cause periodic motion. Flutter occurs on bridges so that a positive feedback occurs between the aerodynamic forces and natural vibration of the bridge. In essence, the vibration movements of the bridge increase the aerodynamic load which in turns cause further movement of the bridge. Consequently, it results in large magnitude of movement and may cause bridge failure.

8. How do vortex-induced vibrations affect the stability of long bridges?

When wind flows around a bridge, it would be slowed down when in contact with its surface and forms boundary layer. At some location, this boundary layer tends to separate from the bridge body owing to excessive curvature. This results in the formation of vortex which revises the pressure distribution over the bridge surface. The vortex formed may not be symmetric about the bridge body and different lifting forces are formed around the body. As a result, the motion of bridge body subject to these vortexes shall be transverse when compared with the incoming wind flow. As the frequency of vortex shedding approaches the natural frequencies of the bridges, resonant vibrations often occur, the amplitude of which depends on the damping in the system and the motion of the wind relative to the bridges. Such oscillations may "lock-on" to the system and lead to hazardous amplification and fatigue failure.

9. What is the difference between gravity anchorage and tunnel

anchorages in suspension bridges?

Gravity anchorages consist of three main parts, namely the base block, anchorage block and weight block. The weight block sits on top of anchor block and its weight is not used for resisting the pull of cables. Instead, its vertical action presses the cables vertically downward so as to turn the pull of cables against the foundation.

Tunnel anchorages resist loads from cables by mobilization of shear friction between the embedded concrete anchorage and the surrounding foundation.

10. What are the advantages of cable-stayed bridges over suspension bridges for span less than 1,000m?

The advantage of cable-stayed bridges lies in the fact that it can be built with any number of towers but for suspension bridges it is normally limited to two towers.

With span length less than 1,000m, suspension bridges require more cables than cable-stayed bridges. Moreover, cable-stayed bridges possess higher stiffness and display smaller deflections when compared with suspension bridges. Generally speaking, the construction time is longer for suspension bridges.

11. When is single plane or multiple plane used in cable-stayed bridges?

For one cable plane to be adopted, the requirement of high torsional stiffness of bridge deck is necessary to enhance proper transverse load distribution. Moreover, owing to the higher stiffness of bridge deck to cater for torsional moment, it possesses higher capacity for load spreading. As a result, this avoids significant stress variations in the stay and contributes to low fatigue loading of cables. On the other hand, the use of one cable plane enhances no obstruction of view from either sides of the bridges.

For very wide bridge, three cable planes are normally adopted so as to reduce the transverse bending moment.

Elastomeric Bearing in Bridges

The use of elastomeric bearing in bridges is not uncommon in Hong Kong. However, not all designers get a full picture of various aspects of elastomeric bearing. Hence, this article is intended to introduce the design of elastomeric bearing to graduate engineers and to refresh the knowledge of experienced bridge engineers.

Basic Design Consideration

For elastomeric bearing, the shear stiffness is an important parameter for design because it influences the force transfer between the bridge and its piers. In essence, elastomers are flexible under shear deformation but it is relatively stiff in compression. However, elastomeric bearings should not be used in tension.

Elastomeric bearing should be designed in serviceability limit state only. The cross sectional area is normally determined by the compressive stress limit under serviceability limit state. The shape factor, i.e. plan area of the laminar layer divided by area of perimeter free to bulge, affects the relation between shear stress and the compressive load. In essence, higher capacity of bearings could be obtained with higher shape factor.

The long side of the bearing is usually oriented parallel to the principle axis of rotation because it facilitates rotational movement. The thickness of bearings is limited and controlled by shear strain requirements. In essence, the shear strain should be less than a certain limit to avoid the occurrence of rolling over at the edges and delamination due to fatigue. Hence, it follows that higher rotations and translations require thicker bearing. On the other hand, the vertical stiffness of bearings is obtained by inserting sufficient number of steel plates. In addition, checks should be made on combined compression and rotation to guard against the possible occurrence of uplifting of corners of bearings under certain load combinations.

Steel Plates Inside the Bearings

In the design of elastomeric bearing, the bearing should be allowed for bulging laterally and the compression stiffness could be increased by limiting the amount of lateral bulging. To increase the compression stiffness of elastomeric bearings, metal plates are inserted. After the addition of steel plates, the freedom to bulge is restricted and the deflection is reduced when compared with bearings without any steel plates under the same load. Tensile stresses are induced in these steel plates during their action in limiting the bulging of the elastomer. This in turn would limit the thickness of the steel plates.

However, the presence of metal plates does not affect the shear stiffness of the elastomeric bearings.

Anchor Dowel

Elastomeric bearing is normally classified into two types: fixed and free. For fixed types, the bridge deck is permitted only to rotate and the horizontal movements of the deck are restrained. On the other hand, for free types the deck can move horizontally and rotate. To achieve fixity, dowels are adopted to pass from bridge deck to abutment. Alternatively, in case there is limitation in space, holes are formed in the elastomeric bearings where anchor dowels are inserted through these holes. It is intended to prevent the “walking” of the bearing during its operation.

Limitation of Elastomeric Bearing

Elastomeric bearing has the potential advantage of low cost when compared with other bearing types. In particular, it requires little long-term maintenance to enhance its performance during servicing. Moreover, it demonstrates good performance in seismic condition because of its relatively large plan areas with low height and the natural dampening effect of elastomer.

In the event of high vertical loads combined with large angle of rotations, rubber bearings are undesirable when compared with pot bearings. For instance, elastomeric bearings require large bearing surfaces so that compression can be maintained between the contact surfaces between the bearings and piers. Moreover, it also leads to uneven distribution of stress on the piers and some of these highly induced stresses may damage the structure. Consequently, pot bearings are better alternatives than elastomeric bearings in such an scenario.

Chapter 2. Concrete Works

1. Is joint filler essential in concrete expansion joints?

The presence of joint filler is essential to the proper functioning of concrete joints though some may doubt its value. For a concrete expansion joint without any joint filler, there is a high risk of rubbish and dirt intrusion into the joint in the event that the first line of defense i.e. joint sealant fails to reject the entry of these materials. In fact, the occurrence of this is not uncommon because joint sealant from time to time is found to be torn off because of poor workmanship or other reasons. The presence of rubbish or dirt inside the joint is undesirable to the concrete structures as this introduces additional restraint not catered for during design and this might result in inducing excessive stresses to the concrete structure which may fail the structures in the worst scenario. Therefore, joint filler serves the purpose of space occupation so that there is no void space left for their accommodation. To perform its function during the design life, the joint filler should be non-biodegradable and stable during the design life of the structure to enhance its functioning. Moreover, it should be made of materials of high compressibility to avoid the hindrance to the expansion of concrete.

2. What is the purpose of setting minimum amount of longitudinal steel areas for columns?

In some design codes it specifies that the area of longitudinal steel reinforcement should be not less than a certain percentage of the sectional area of column. Firstly, the limitation of steel ratio for columns helps to guard against potential failure in tension. Tension may be induced in columns during the design life of the concrete structures. For instance, tension is induced in columns in case there is uneven settlement of the building foundation, or upper floors above the column are totally unloaded while the floors below the column are severely loaded. Secondly, owing to the effect of creep and shrinkage, there will be a redistribution of loads between concrete and steel reinforcement. Consequently, the steel reinforcement may yield easily in case a lower limit of steel area is not established.

In addition, test results showed that columns with too low a steel ratio would render the equation below inapplicable which is used for the design of columns:

$N=0.67f_{cu}A_c+f_yA_s$

3. What is the purpose of reducing the seasonal and hydration temperature by one-half in the calculation of crack widths arising from thermal movement?

In the calculation of thermal movement, the following formula is used in most codes:

w_{max}=s x a x (T_1+T_2)/2

where w_{max} = maximum crack width
s = maximum crack spacing
a = coefficient of thermal expansion of mature concrete
T_1= fall in temperature between peak of hydration and ambient temperature
T_2= fall in temperature due to seasonal variation

For T_1, it represents the situation when the freshly placed concrete is under hydration process. Since the occurrence of high creep strain to the immature concrete tends to offset the effect of early thermal movement, a factor of 0.5 is purposely introduced to take into account such effect.

For T_2, it refers to the seasonal drop in temperature for the mature concrete. Owing to the maturity of concrete in this stage, the effect of creep on concrete is reduced accordingly. Since the ratio of tensile strength of concrete (f_{ct}) to average bond strength between concrete and steel (f_b) increases with maximum crack spacing, the lower values of f_{ct}/ f_b in mature concrete leads to smaller crack spacing. Therefore, the increased number of cracks helps to reduce the effect of thermal movement brought about by seasonal variation. Hence, T_2 is reduced by one-half to cater for further creep and bond effects in mature concrete.

4. Should mild steel or high yield steel be adopted as reinforcement of water-retaining structures?

In designing water-retaining structures, movement joints can be installed in parallel with steel reinforcement. To control the movement of concrete due to seasonal variation of temperature, hydration temperature drop and shrinkage etc. two principal methods in design are used: to design closely spaced steel reinforcement to shorten the spacing of cracks, thereby reducing the crack width of cracks; or to introduce movement joints to allow a portion of movement to occur in the joints.

For the choice of steel reinforcement in water-retaining structures, mild steel and high yield steel can both be adopted as reinforcement. With the limitation of crack width, the stresses in reinforcement in service condition are normally below that of normal reinforced concrete structures and hence the use of mild steel reinforcement in water-retaining structure will suffice. Moreover, the use of mild steel restricts the development of maximum steel stresses so as to reduce tensile strains and cracks in concrete.

However, the critical steel ratio of high yield steel is much smaller than that of mild steel because the critical steel ratio is inversely proportional to the yield strength of steel. Therefore, the use of high yield steel has the potential advantage of using smaller amount of steel reinforcement. On the other hand, though the cost of high yield steel is slightly higher than that of mild steel, the little cost difference is offset by the better bond performance and higher strength associated with high yield steel.

5. How are shear forces transmitted across a reinforced concrete crack?

There are two principle mechanisms in transferring shear forces across a reinforced concrete crack, namely, aggregate interlock and dowel action. The aggregate interlock refers to the interaction between rough surfaces of the crack. The shear stiffness of aggregate interlock is influenced by the axial tensile stiffness of the reinforcement. When shear displacement occurs, there is a tendency for the crack to widen. In the meantime, the reinforcement restrains the crack widening, which subsequently increases in axial stress of the reinforcement. On the other hand, dowel action refers to the shear resisted by the reinforcement.

6. What is the difference between foam concrete and cement grout?

Foam concrete is mainly composed of cement, water and air pores with filler (such as PFA, sand etc.) without any course aggregates. The air pores are formed by agitating air with a foaming agent. The typical size of air bubbles is around 0.3-0.4mm in diameter. For cement grout, it mainly consists of cement and water.

Foam concrete is characterized by have low density and low cost when compared with normal concrete. The density of foam concrete is around 400 – 1600 kg/m^3. Therefore, the low density enhances low dead load and has extensive applications when low loadings are required. Foam concrete does not require compacting and hence imposes no lateral forces on adjacent structures. Moreover, it also displays good resistance to water and produces high level of sound and thermal insulation. However, it suffers from the demerit that it have low compressive strength only (e.g. less than 15 MPa) which is drastically different from cement grout which possesses high compressive strength. There is recent development of foam concrete as road sub-base.

The Use of Ground Granulated Blast Furnace Slag (GGBS) in Concrete

Abstract:

The use of ground granulated blast furnace slag (GGBS) as cement replacement is prevalent in many countries like Japan and the Mainland. However, there is only limited reported usage of GGBS in Hong Kong. In fact, there are various technical issues and potential problems like the source of material supply associated with GGBS which discourage engineers to adopt such material.

This paper studies the major properties of ground granulated blast furnace slag in detail, followed by the discussion of the potential demerits associated with this cement substitute leading to its limited usage in local market. It serves as a reference to engineers who are involved in the design and selection of concrete mixes.

Keywords: Slag, Heat of Hydration, Sulphate Attack, Chloride Attack, Curing, Bleeding, Cracking

Introduction

Ground granulated blast furnace slag is renounced of its capability to replace cement up to 75% level and this brings about environmental benefits due to the drastic reduction of the usage of cement in concrete production. From technical point of view, the use of GGBS also improves the durability of the concrete mixes owing to the reduced heat of hydration and better resistance against chloride attack and sulphate attack. Notwithstanding the environmental and technical benefits associated with GGBS, the use of GGBS in local market is quite limited in application. This paper serves to investigate the merits and demerits of GGBS and explore the potential usage of GGBS in local market.

Merits of GGBS

Lower Heat of Hydration

The amount of heat of hydration affects the long-term durability of concrete and it is desirable to keep it to a low value. The heat of hydration, when subject to restraint, gives rise to external or internal cracking which impairs the durability of concrete. There are two forms of restraint, i.e. external and internal. Let's take a circular column as an example to illustrate the effect of the heat of hydration with internal restraint: When the temperature is rising, the inner concrete's temperature is higher than outer concrete's temperature and the inner concrete is expanding. This induces pressure to the outside and the induced compressive stress will result in the formation of radial cracks near the surface of concrete. On the other hand, when the temperature drops, the concrete at the outside drops to surrounding temperature while the concrete at the central region continues to cool down. The contraction associated with inner concrete induces tensile strains and forms cracks tangential to the circular radius. For external restraint, when the concrete section is fully restrained at its ends and it is prevented from contracting when the temperature drops as a result of heat of hydration. Consequently, tensile stress induced in the member cause the cracking across the its cross section.

Figure 1 – Diagrams showing crack formation during rise and fall of hydration temperature

The partial replacement of cement with GGBS enhances the reduction of heat of hydration. Based on previous studies, the difference between temperature rise of ordinary Portland cement concrete and of 50% slag concrete and 75% slag concrete is about 1.0^{o}C per 100kg and 2.0^{o}C per 100kg respectively. A progressive reduction in maximum temperature rise was obtained with increasing slag replacement percentage. At 30% slag replacement, a modest reduction in temperature rise of 5% was obtained when compared to normal OPC concrete. However, the reduction of temperature rise for 50% and 70% slag replacement was at 10% and 20% respectively (Table 1).

Slag replacement level	50%	70%
Reduction of temperature rise in hydration	10%	20%

Table 1 – Comparison of the effect of different slag replacement level on the reduction of hydration temperature

Other than the merit of lower heat of hydration, the replacement of OPC with GGBS increases the time to reach peak temperature and this also benefits the concrete structure from durability point of view.

Enhanced Durability

The durability of concrete is enhanced by the introduction of slag to replace part of cement. The increase of durability is attributed by the following reasons:

(i) Other than its primary reaction with water, GGBS also reacts with calcium hydroxide (generated from hydration of tricalcium silicate and dicalcium silicate) to form cementitious material. During this process, the amount of calcium hydroxide is greatly reduced which is vulnerable to chemical attack.

(ii) The particle size of GGBS is small when compared with cement particles. With increasing amount of GGBS the pores become smaller. For slag concrete it contains more gel pores and fewer capillary pores than normal concrete and thus contributes to the reduction of permeability.

(iii) Slag concrete possesses higher resistance to ionic diffusion than normal concrete and the resistance is found to increase with GGBS percentage.

The durability of GGBS concrete is higher than normal OPC concrete with equivalent grade. This is due to the continued hydration of GGBS which reduces the porosity and permeability of the paste structure and to the modified chemistry of the cement paste. The durability of slag concrete is governed by the volume and distribution of pores in the cement paste. Initially the capillary pre system is full of sulphate ions from the gypsum, calcium ions from the cement, alkali and hydroxyl ions. When slag commences dissolving in the alkaline solutions, it forms secondary calcium silicate hydrates which fills capillary pore space left by the first hydration of cement. As a result, the hydration of GGBS renders the capillary pores discontinuous by providing secondary blockages of calcium silicate hydrates across pores. Moreover, GGBS has an important role in increasing the bond and decreasing porosity of cement-aggregate interfaces in concrete.

Resistance to Sulphate Attack and Chloride Attack

For sulphate attack, there are mainly two reactions involved. The first one is the reaction between sulphate ions and hydrated calcium aluminate and it results in the formation of ettringite. This is the main reaction responsible for normal sulphate attack. The second one is the combination of sulphate ions with calcium hydroxide forming gypsum. In essence, the degree of sulphate attack is controlled by the amount of calcium aluminate and the alumina content of GGBS. It was reported that for slag concrete with replacement more than 65%, sulphate resistance was always greater than that of normal concrete.

Slag concrete is more resistant to chloride attack than normal concrete on the basis of equivalent cement content or grade. In particular, GGBS concrete is renounced of its durability in maritime environment. Moreover, with addition of slag in concrete, the diffusion rates of both chlorides and oxygen are reduced so that the time to depassivation is extended. Moreover, the lower calcium/silicon in calcium silicate hydrates developing from the slag increases chloride-binding ability and aid in reducing the rate of chloride ingress.

Environmental Benefits

GGBS is a by-product of steel industry – slag is produced at the same time as pig iron; subsequently granulated by rapid quenching and ground to desired fineness as cementitious material. Since GGBS is always produced during the manufacture of steel, there is only some energy input to grind and process the slag to finer sizes and this is considered as highly environmentally friendly.

As a rough estimate, one tonne of carbon dioxide is discharged into the atmosphere for every tonne of cement manufactured. Therefore, the use of GGBS as replacement of cement greatly reduces the amount of cement employed as ingredients of concrete. In particular, GGBS is considered as a better choice than PFA from environmental protection point of view because the level of replacement of GGBS is up to about 75% which is far higher than the upper limit of PFA replacement (40% replacement).

Potential Problems of GGBS

Bleeding

Bleeding is a potential hurdle for the widespread use of GGBS in concrete. Based on various literatures on the behavior of GGBS in terms of bleeding, the higher proportion of GGBS to be used, the more likely bleeding would occur due to a delayed setting time.

Bleeding is a form of segregation and is the process of sinking of solids with displacement of water upwards within the fresh cement paste. During the bleeding process, channels or water lenses under aggregates might be formed and they would impair the durability of the concrete. On the other hand, the fine particles it carries during bleeding produce a weak laitance layer on the top surface of concrete. Hence, the process of bleeding poses problems to the concrete during its servicing life.

Bleeding of the concrete with slag content was time-dependent. For instance, the bleeding of concrete with higher slag content was less than that of OPC concrete during mixing and during first 40 minutes after mixing. The amount of bleeding water decreases with increasing fineness of the cements. Take GGBS produced in Britain as an example to illustrate the typical fineness of GGBS and cement. The fineness of GGBS is about 400-450m^2/kg while the fineness of cement is 350m^2/kg. Therefore, the finer GGBS demands more water during the hydration process, leading to theoretically reduction in bleeding.

However, after 40 minutes those concrete with higher slag content tend to bleed more water. This time-dependent behaviour of GGBS was attributed to the rapid hydration, the higher surface area and the angular shape of GGBS. When GGBS reacted, it serves as a fine mineral aggregate and thus reducing the requirement of water. Hence, after 40 minutes the amount of bleeding increased substantially.

The amount of bleeding is also related to the amount of GGSB used in the concrete. With higher slag replacement (i.e. higher than 25%) it would result in concrete with increased bleeding. In essence, cement with blast-furnace slag often exhibits a delayed setting time, leading to an increased period of bleeding.

Strength Development

Slag concrete develops strength more slowly than OPC concrete at normal temperatures. The rate of gain of strength at early ages for slag concrete increases far more with an increase in curing temperature than an equivalent OPC concrete. Under adiabatic conditions concrete containing 70% slag had reached the strength of the equivalent OPC concrete by 7 days. The reaction of slag tends to retard the reaction of cements at early ages. This may be due to the reaction of slag which used up part of the calcium hydroxide formed during earlier ages of hydration. On the other hand, the glassy compounds in GGBS react slowly with water and it takes time to obtain hydroxyl ions from the hydration product of cement to split the glassy slag particles in early age. Therefore, it explains the relative slow reaction rate of hydration for GGSB concrete in early age.

Though the presence of GGBS lowers the rate of hydration leading to lower initial concrete strength, the use of GGBS possesses the advantage of higher ultimate concrete strength based on previous experimental results. Addition of slag in concrete up to 75% increases the compressive strength of concrete by 28% when compared with concrete without slag.

To overcome the low strength development associated with GGBS, the early strength of slag concrete can be achieved by further grinding of

slag, high temperature curing or using low water cement ratio.
Sensitivity to Curing

Strength development and durability of GGBS concrete are both highly susceptible to curing conditions, more so than comparable grade of OPC concrete. Since the early strength development would be slow, the curing time for GGBS concrete would likely require a longer period than those of Portland cement or PFA concrete. Moreover, the sensitivity to curing of GGBS concrete is attributed to the reduced formation of hydrates in early ages leading to increased loss of water which would otherwise be available for hydration to continue.

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Past experimental works were carried out to compare the effect of curing to OPC concrete and slag concrete. The results showed that slag concrete was far more sensitive to increases in temperature than equivalent OPC concrete. The rate of gain of strength at early ages for slag concrete increased far more with an increase in curing temperature than an equivalent OPC. Similar to Portland cement concrete and PFA concrete, there is an optimum temperature beyond which an increase in curing temperature would give rise to lower strength and other properties. The optimum temperature for PFA concrete and GGBS concrete are about 30 ºC and 50 ºC respectively.

Though GGBS concrete was proven to be sensitive to curing conditions, the degree of sensitivity depends on the size of the concrete. Thin sections of GGBS concrete are sensitive to curing in drying conditions. However, thicker structural section may be less sensitive to curing. This is attributed to the rise in temperature during initial hydration and this is significant at the surface due to the use of formwork and surplus moisture in the larger mass of concrete.

To take into consideration the slow hydration rates of GGBS which accounts for its sensitivity to curing conditions, BS8110:1985 recommends extended curing time for GGBS concrete in adverse conditions.

Conclusion:

This article presents various associated with the use of GGBS as partial cement replacement. From structural point of view, GGBS replacement enhances lower heat of hydration, higher durability and higher resistance to sulphate and chloride attack when compared with normal ordinary concrete. On the other hand, it also contributes to environmental protection because it minimizes the use of cement during the production of concrete.

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Joints in Concrete Structures

Concrete expands when heated and contracts when cooled. If insufficient joints are provided along concrete structures, the restraint against movement would generate stresses which may crack and cause the malfunctioning of the structure. In this connection, careful consideration and design of concrete joints is of utmost importance to enhance the durability of the concrete structure and this articles serve to provide a clear picture of the functions of different component of concrete joints and to offer the information required during the design of concrete joints.

Joint Sealant

Most joint sealants applied in concrete joints are adhesive and the recommended joint width/depth of joint sealant is from 2:1 to 1:1 as given by BS6213 and Guide to Selection of Constructural Sealants. Joint sealants perform by closing the joint width and preventing water and dirt from entering the joint and causing dowel bar corrosion. If rubbish or dirt is trapped inside the joint, additional joint stress during the expansion movement of concrete structure would be resulted from restrained movement.

When joint sealant is applied on top of joint filler in concrete joints, additional primers are sometimes necessary because primers help to seal the surface to prevent chemical reaction with water on one hand. On the other hand, it provides a suitable surface for adhesion of joint sealant.

Designers have to take into account the movement accommodation factor during the design of joints of concrete structures. Movement accommodation factor is commonly specified by manufacturers of joint sealants for designers to design the dimension of joints. It is defined as the total movement that a joint sealant can tolerate and is usually expressed as a percentage of the minimum design joint width (CIRIA Special Publication 80 (1991)). Failure to comply with this requirement results in overstressing the joint sealants.

For example, if the expected movement to be accommodated by a certain movement joint is 5mm, the minimum design joint width can be calculated as 5 ÷30% = 16.6mm where the movement accommodation factor is 30%. If the calculated joint width is too large, designers can either select another products of joint sealants with higher movement accommodation factor or to redesign the layout and locations of joints.

Bond Breaker

Joint sealant should be designed and constructed to allow free extension and compression during the opening and closure of joints. In case joint sealants are attached to the joint filler so that movement is prohibited, they can hardly perform their intended functions to seal the joints against water and debris entry. Hence, some form of bond breaker is introduced in concrete joints to separate joint sealant and joint filler so as to eliminate the stresses arising from the adhesion of joint sealant to joint filler in case bond breaker is not provided. Polyethylene tape is commonly used as bond breaker tape.

To facilitate free movement, it can be achieved by adding bond breaker tape in-between the joint sealant and joint filler. Primers may be applied to the sides of joints to provide a good bond between them.

Joint Filler

The presence of joint filler is essential to the proper functioning of concrete joints though some may doubt its value. For a concrete expansion joint without any joint filler, there is a high risk of rubbish and dirt intrusion into the joint in the event that the first line of defense i.e. joint sealant fails to reject the entry of these materials. In fact, the occurrence of this is not uncommon because joint sealant from time to time is found to be torn off because of poor workmanship or other reasons. The presence of rubbish or dirt inside the joint is undesirable to the concrete structures as this introduces additional restraint not catered for during design and this might result in inducing excessive stresses to the concrete structure which may fail the structures in the worst scenario. Therefore, joint filler serves the purpose of space occupation so that there is no void space left for their accommodation. To perform its function during the design life, the joint filler should be non-biodegradable and stable during the design life of the structure to enhance its functioning. Moreover, it should be made of materials of high compressibility to avoid the hindrance to the expansion of concrete.

Dowel bar

This is a major component of the joint. It serves to guide the direction of movement during concrete movement. Therefore, incorrect direction of placement of dowel bar will induce stresses in the joint during thermal expansion. On the other hand, it connects the two adjacent structures structurally by transferring loads across the joints.

Chapter 3. Drainage and Tunneling Works

1. What is the purpose of carrying out water absorption test for precast concrete pipes?

Cement will mix with more water than is required to eventually combine during hydration of cement paste. As such, some voids will be left behind after the hydration process which affects the strength and durability of concrete. With the presence of air voids in concrete, it is vulnerable to penetration and attack by aggressive chemicals. Good quality concrete is characterized by having minimal voids left by excess water and therefore, water absorption test for precast concrete pipes is adopted for checking the quality of concrete in terms of density and imperviousness.

2. Which types of soils are unsuitable for testing under sand replacement test?

Any soils that can be excavated with handtools is suitable provided that the void or pore openings in the soil mass are small enough to prevent the calibrated sand used in the test from getting into the natural voids. Moreover, the soils being tested should have sufficient cohesion or particle interlocking to maintain side stability during excavation of the test pit. Furthermore, it should also be firm enough not to deform or slough due to the pressures exerted in digging the hole and pouring the sand.

3. What is the difference between road gullies and catchpits?

Both road gullies and catchpits are the two basic types of drainage inlets of drainage system. Though they are designed to catch stormwater, road gullies and catchpits are intended to catch stormwater at different locations. Catchpits are designed to receive stormwater from slopes and stream courses. There is no standard design of catchpits and they can take different forms and shapes like inclusion of sand trap to improve the quality of collected stormwater and to prevent the blockage of drains. On the other hand, road gullies are intended to receive stormwater from roads only.

4. What is the difference between on-line storage and off-line storage in the design of storage pond?

The design of storage pond is commonly divided into on-line storage and off-line storage. The on-line storage concept involves the inclusion of storage facilities in series with the pipelines so that overflow at the storage facilities is allowed. One simple application of on-line storage is to enhance a large size of drainage pipes. However, for heavy rainfall situation, the spare capacity of drainage pipes will be rapidly exhausted. On the other hand, off-line storage (e.g. underground storage tank) refers to storage facilities in parallel with the pipeline and the return flow to the main pipeline is only allowed when the outflow pipelines are not surcharged.

7. Does Moody Diagram used for calculating energy losses in pipes suitable for all conditions?

Darcy-Weisbach equation combined with the Moody Diagram is the accepted method to calculate energy losses resulting from fluid motion in pipes and other closed conduits. However, the Moody Diagram may not be suitable for usage in some conditions. For instance, the curve at transition region between laminar and fully turbulent rough pipe flow is applicable for pipes with interior roughness comparable to iron. Moreover, owing to the difficulty in determining pipe roughness, the accuracy of Moody Diagram is only about plus or minus 15%.

8. Why are dimples present in golf balls?

The golf balls are subject to lift and drag forces when struck. The drag force is a retarding forces acing in a direction opposite to the direction of flight path of the golf ball. Separation occurs behind the ball which forms a low-pressure wake. The pressure drag is the difference in pressure between the front and back of the ball.

With dimples on the ball, the formation of separation is delayed so that it narrows the size of wake. Consequently, less pressure is pulling on the back of the ball so that the golf ball could travel longer in distance.

9. How can porous pavement help to relieve flooding problem?

Common structural approaches to combat flooding problem includes replacement/upgrading of drains, temporary flood storage with pumping facilities, cross catchment diversion and infiltration. The concept of infiltration involves the reduction of surface runoff so that the amount of overland flow is reduced. To enhance absorption of runoff into the ground, structures and facilities have to be designed and provided. For instance, porous pavement and infiltration pond could serve as infiltration facilities to cut off some surface runoff.

10. What is "residual flooding"?

It is not uncommon that local flooding still occurs despite that drainage improvement measures have already been implemented in its vicinity. In some low-lying areas the ground level is lower than the water level in the nearby main drainage channel. As the channel collects stormwater from catchment by gravity, it is natural to follow that runoff from these low-lying areas can hardly be discharged into the channels during rainstorm. Such phenomenon is called "residual flooding".

11. How to achieve flood prevention by On-site Stormwater Detention?

The concept of On-site Stormwater Detention involves the temporary storage of stormwater with limited runoff from the site. It essentially modifies the runoff behaviour of the site so as to prevent flooding in area downstream. The allowable rate of discharge from the given area is Permissible Site Discharge while the minimum storage area for water detention is called Site Storage Requirement.

Examples of storage facilities for Permissible Site Discharge are flat roof, underground pits, fish ponds etc.

12. What is the difference between pipe jacking and micro-tunneling?

Pipe jacking is a general technique of the installation of pipes with a tunneling shield in front and the pipes are jacked from a jacking pit to a receiving pit. The tunneling shield for pipe jacking can be electrical and mechanical equipment for conducting the excavation work or it can be a manual shield for workers going inside the shield to carry out manual excavation. For microtunneling, it is a kind of pipe jacking of small sized non-man-entry pipes which are remotely controlled. In general, there are two common types of micro-tunneling machines:

(i) Pressurised slurry
Similar to the Pressurised slurry TBM, excavated material is transported from the excavation face to the surface suspended in a slurry.

(ii) Auger machine
Excavated material is transported from the excavation face to the drive pit through a cased screw auger.

13. Why does the problem ground settlement occur when pipe-jacking machine enters mixed ground with soils and boulders?

The rate of cutting through soils is faster than that of cutting through boulders for pipe-jacking machine. As such, when pipe-jacking machine enters a region of mixed ground with soils and boulders, the machine has the tendency to move towards the direction of soft soils because of the difference of rate of advancement of pipe-jacking machine for soils and boulders. Consequently, migration of soft soils occur which contributes to ground settlement. The degree of settlement is dependent of the depth of soil cover, soil property and the size of boulders.

14. Why does pipejacking machine usually get stuck when the ground condition change from soil to very hard rock?

When the pipejacking machine moves from a region of soil to very hard rock, it will be subject to damage of cutting disc. To break and loosen the rock, the pipejacking machine applies a large torque on cutting wheels. However, with the change of soft region to hard region, the pipejacking machine is still under the same jacking load. As such, this results in insufficient or little space for the movement of the machine against the rock face, leading to damage and exhaustion of the pipejacking machine.

15. Would ground settlement occur ahead or behind the jacking face for pipe-jacking?

It is reported by Lake (1992) that settlements are expected at the ground surface at a distance of 1-2 times of tunnel depth ahead of the tunnel face and 80% to 90% of settlement to be completed at a similar distance behind the face. However, in the paper "Monitoring of ground response associated with pipe jacking works – recent experience in Hong Kong", the author pointed out that based on their experience, development of longitudinal settlement was observed at a distance of 3-4 times of tunnel depth behind the tunnel face and little settlements were reported immediately above the tunneling face.

16. Can shotcrete be adopted as permanent linings in tunnels?

Shotcrete can be employed as permanent linings in tunnels. However, it is only expected that they would not carry loads in eth same way as structural linings. The use of shotcrete is limited to temporary works in tunnels with fair good ground conditions which is self-supporting. Its presence is used for avoiding rock falls or erosion problems.

17. What are the major factors affecting ground-support interaction in tunnels?

In general, the in-situ ground stress reduces with an increase in inward radial displacement in an unloaded ground/tunnel. The major factors affecting ground-support interaction in tunnels are as follows:

(i) The relative stiffness of ground and linings plays an important role in ground-support interaction. For instance, a stiff support could support the unloaded ground at lower deformation than a flexible support.

(ii) Delay of support to tunnels results in the ground to be deformed by a certain degree before the installation of linings. Consequently, the linings take up less loads subsequently owing to less deformation of ground generated after the action of delay of support.

(iii)As the ground is unloaded, stress is redistributed during excavation. In case the ground is delayed in support and the ground respond from elastic region to yielding region, it results in larger displacement and supporting load.

Watertightness in Joints of Drainage Channels and Box Culverts

The question about the necessity of watertightness in joints of drainage channels is often raised by many graduate engineers and I understand that this question has troubled most graduate engineers during the design of drainage channels and box culverts. In particular, I was approached by many graduate engineers and assistant engineers regarding the need of the provision of waterstop in joints of drainage channels. In this connection, this article serves to provide a clear picture of the issue, thereby eliminating their doubts and queries when graduate engineers come across this issue again during their day-to-day design work.

In designing the joints for box culverts and drainage channels, the requirement of watertightness is usually adopted to maintain water flow in channel and it is normally implemented by the installation of waterstop across the joints of the drainage structure. The plain dumb-bell type waterstop is used for joint location where small movements are anticipated. Therefore, construction joints are desirable locations of this type of waterstop. On the other hand, center-bulb type waterstop is suitable for expansion joints or locations where lateral and shear movements occur due to settlement or deflection.

It is common engineering practice to design weepholes along the walls of drainage channels to relieve the water pressure so that the effect of water pressure can be ignored during the design of walls of channels. By considering the effect of the presence of weepholes, it is natural to follow that the watertightness achieved by waterstop across the joints appears to be meaningless because water flowing in channels can escape through the weepholes on the walls of the drainage structures. In case the purpose of maintaining watertightness in joints of drainage structures is to reduce the amount of water loss through joints, the above argument seems to be correct. However, the main reason of achieving watertightness across joints in drainage structures is not directly related to the amount of water loss in drainage channels and box culverts.

Watertightness in joints of drainage channels and box culverts is considered desirable for the following two scenarios:

(i) Where there is a high possibility of occurrence of high water table in the vicinity of box culverts/channels, waterstop would be installed across the joints. The high groundwater level and rainwater seepage through embankment may cause water passing through the joints and washing in soils. Consequently, the loss of soils may lead to the failure of the structures when subject to severe loads on the drainage structures.
(ii) In case the box culvert/channels are designed in such a way that water flow through joints from the structures causes the washing out of bedding materials, the requirement of watertightness of joint has to be fulfilled. Otherwise, similar consequence of structure failure may result as mentioned above when the box culvert/channels experience heavy imposed/traffic loads.

Other than the concern of washing of foundation material, watertightness in joints of drainage channels and box culverts is considered beneficial because it may have the added benefit of preventing the corrosion of dowel bars across the joints. In essence, dowel bars serve to maintain the alignment of the structure when the structure experiences contraction and expansion throughout the design life on one hand; and to transfer shear forces across the joints on the other hand. In the event that the corrosion of dowel bars occurs leading to its malfunction, the joints would be subject to additional stresses which may lead to its failure. For instance, if it is anticipated that the drainage channel/box culverts would be continually in contact with seawater which contains various corrosive agents, it is recommended that watertightness of the joints should be achieved in order to avoid the corrosion problem of dowel bars in the joints. In cold countries where road salt is sometimes applied on roads above box culvert or at crossings of channels to prevent freezing and thawing, the leaching of road salts into the joints may cause corrosion of joint reinforcement which may also be minimized by adoption of watertight joints.

Chapter 4. Marine Works

1. In reclamation involving large volumes of fill and tight programme, shall engineers use marine fill or mud extracted from land borrow area as filling material?

There are two advantages of adopting marine fill over mud extracted from land borrow area:

(i) In some land borrow areas, it involves breaking up of rock to suitable sizes for reclamation and the production rate is not high. With modern equipment for dredging and placing marine fill, the filling rate is much higher.

(ii) The cost incurred for breaking up of rock to suitable sizes for reclamation is very expensive while the cost of hydraulic filling with marine fill is lower.

2. What are the factors determining the stability of a single armour unit?

There are mainly three main factors which govern the stability of a armour unit, namely, gravity, intertangling and squeezing. Obviously, it is beyond doubt that the ability of the armour unit to stay in place should be closely related to gravity force. On the other hand, the geometry of the armour unit also affects its stability. For instance, with the difference in ability to intertangle, their resistance to pulling out by waves varies. Furthermore, squeezing forces by gravity also affects the stability of the armour unit which is dependent on frictional forces in all directions.

3. Hudson's formula and Van der Meer formula are commonly used in the design of armour. Which one is a better choice?

Hudson's formula is commonly adopted in preliminary design to obtain rough initial estimate of rock size. The formula is derived from the results of regular wave tests. However, this formula does not take into account the following elements which Van der Meer formula does:

wave period, damage level, permeability of structure and storm duration. Moreover, Hudson's formula deals with the use of regular waves only.

Compared with Hudson's formula, Van der Meer formula is more complicated and it is derived from results of a series of physical model tests. They include the consideration of wave period, storm duration, clearly-defined damage level and permeability of structure. The choice of the appropriate formula is dependent on the design purpose (i.e. preliminary design or detailed design).

4. Why are observed settlements in reclamation normally larger than calculated?

Settlement in reclamation area occurs as a result of primary consolidation (i.e. by dissipation of excess pore water pressure) and secondary compression which involves creep of soils. Creep of soils occurs by viscous squeezing out of absorbed water in double layers of clay particles and rearrangement of clay particles under loading.

In the calculation of settlement in reclamation, there are generally two methods available. The first method assumes that creep occurs during the process of primary consolidation, which appears to be logically correct. On the other hand, the second method assumes that creep of soils occurs after primary consolidation and hence at the end of consolidation period the calculation settlement is equal to primary consolidation only without any consideration of creep effects. This method of settlement calculation is well adopted by most consulting firms and hence leads to underestimation of total settlement.

5. Vibrocompaction is carried out to loose sand after reclamation by filling sand. How to determine the in-situ density of filled sand?

It is difficult to obtain undisturbed samples of sand to measure the in-situ density. The relative density of in-situ sand can be determined by correlation to other parameters in some methods. For instance, Cone Penetration Test can be employed to estimate the relative density of sands as there is well established relation between cone tip resistance and relative density.

On the other hand, shear wave velocity measurements using seismic CPT and Spectral Analysis of Surface Waves can be used to obtain relative density indirectly.

6. Why do compliance test be carried out some time after the completion of vibrocompaction?

A process called "ageing" occurs after the operation of vibrocompaction. It is observed that sand fill shows an increase in strength and stiffness. Upon immediate completion of vibrocompaction, the apparent increase in strength of sand fill is due to dissipation of air and water pressure. Then the ageing process shall continue owing to creep and cementation.

Hence, sufficient time shall be established for ageing of sand fill to develop prior to compliance testing (such as CPT test). Otherwise, it may result in non-compliant strength and requires re-compaction.

9. For reclamation by hydraulic filling with sand, it is commonly observed that the density of sand varies significant with depth. Why?

In reclamation works, the density of filled sand varies with the method of placement. For example, bottom dumping is adopted for sand as the method of placement and this results in higher density of filled sand. For placement by pipeline discharge, the density of filled sand formed by this method is lower than that by bottom dumping. Hence a weak zone is formed for sands placed by pipeline discharge and hence these loose sand fill (relative density about 30%) may cause settlement when subjected to loading or vibrations.

In general, the filled sand above water table is found to be higher in density as they are well compacted by bulldozers and traffic of constructional plant.

10. How does overflowing in trailing suction hopper dredger affect the water regime?

Trailing suction hopper dredger contains a large hopper for storage and transport of dredged materials. The dredging operation is implemented by a hydraulic dredging system including draghead, suction pip, pumps for taking up the material from seabed and putting them into the hopper.

When dredging materials, overflowing may occur to increase the solid loads in the hopper and improve the efficiency of dredging operation. The removal of excess water and soil/water mixture with low density enhances the storage of soils mixture with higher density. Hence, this lowers the cost of dredging by increasing the rate of production. If no overflow is allowed during the dredging operation, trailing suction hopper dredger can normally carry about 10% of normal load and this essentially increases the cost of dredging operation.

If overflowing is allowed in the dredging operation, it shall work on unrestricted basis as the rate of overflow is nearly constant over the entire overflowing process. For dredging with overflowing, the particle size distribution of sand may differ from the in-situ grading because overflowing tends to remove the fine content of sand.

The loss of sediment associated with overflowing poses environmental problems to the nearby water zone in eth following ways:

(i) increased sedimentation
(ii) decrease in dissolved oxygen
(iii) increased turbidity
(iv) increased amount of nutrients

11. Why can vacuum preloading be employed to accelerate the rate of consolidation?

In vacuum preloading, the drainage boundary of clay is isolated from the atmosphere by a membrane. A partial vacuum (e.g. suction of 80kPa) is applied within the membrane to reduce the water pressure so as to speed up consolidation.

The rate of consolidation can be increased by surcharge preloading in which the excess pore water pressure in clay is temporarily increased. Alternatively, the rate of consolidation can also be increased by vacuum preloading by a decrease in water pressure.

Vacuum preloading is generally faster in operation than surcharge preloading which requires timely delivery of fill on top of clay. Moreover, it is unnecessary to consider the stability criterion which surcharge preloading should require.

12. Why do prefabricated band drains become more popular than sand drains?

There is a trend that sand drains are replaced by prefabricated band drains due to:

(i) The rate of installation of prefabricated band drains is high. As such, substantial cost and time could be saved by using this method of construction.
(ii) The disturbance of soil during installation of drains is smaller when using prefabricated band drains.
(iii) The ease of fabrication, quality control and storage associated with band drains.

13. What is the purpose of formation of bunds in reclamation?

Reclamation works normally proceed behind the seawall to protect against typhoon attack. In case where soft marine mud is encountered during reclamation, bunds may be formed on planned alignment of road and drainage works and locations of early development to displace mud to other less important areas should mudwaves indeed occur. Reclamation may be carried out in strips or even crossed bunds forming a grid. Hence, if mudwaves really occur, they could be isolated and dealt with individually.

A Review on the Use of Marine Piling System of Steel Tubular Piles with Reinforced Concrete Infill

Introduction

The marine piling system of steel tubular piles with reinforced concrete infill is used widely in marine structures like piers, jetties, dolphins, bridges etc. The construction of this piling system involves driving steel tubular piles into seabed by hammers with the formation of soil plug in the lower position of hollow space of the piles, followed by placing of reinforcement cage inside the piles and concreting the piles up to the level of pile caps. A typical arrangement of the piling system is shown in Figure 1. This system was prevailing in the past owing to its relatively lower construction cost when compared with other piling methods like bored piles. However, with increasing usage of this type of marine piling system, some drawbacks and problems associated to this piling system have come into light.

The Marine Piling System of Steel Tubular Piles with Reinforced Concrete Infill

Steel tubular piles

The use of steel tubular piles with reinforced concrete infill is commonly adopted as marine piles. This type of marine piling system mainly consists of steel tubular piles driven to a certain depth below seabed based on set criterion. Tubular piles are commonly adopted instead of other geometrical shapes (e.g. H-piles) because it has the potential advantage of higher column-buckling strength when compared with others. The higher is the second moment of area of a steel section, the higher is the column-buckling load (Table 1). Since marine piles normally stand well above seabed, higher buckling strength associated with circular piles enhances efficiency in the design of piles. Moreover, circular piles possess higher energy absorbing capacity which has versatile application in maritime structures like jetties and dolphins which require substantial amount of berthing energy to be absorbed.

	Circular Hollow Sections Outer Diameter 355.6mm with 12.5mm thickness	Universal Bearing Piles 356x368x109 with 12.8mm thickness
Second Moment of Area	19850cm^4	10990cm^4

Table 1 – Comparison of second moment of area between Circular Hollow Sections and Universal Bearing Piles of similar size

Normally a layer of protectively coating like polyethylene is designed on the external face of tubular piles. Since the piles are located in marine environment with direct contact with seawater, the protective coating serves to reduce the chloride attack on the marine piles leading to steel corrosion. However, during the pile driving process of steel tubular piles it stands a high probability that the external coating below seabed level would be impaired. Therefore, in the design of steel tubular piles it is normally assumed that the piles above seabed level are considered to be ignored or sacrificial taking into account the high possibility of corrosion of steel in marine environment within the design life of 50 years.

Reinforced Concrete Infill

Reinforced concrete is designed to fill the void space inside the steel tubular piles from pile cap to a certain distance below seabed. As mentioned earlier, steel tubular piles above seabed level is assumed in design to be completely corroded when approaching the end of design life. As such, loads from pile caps are transferred directly to reinforced concrete infill instead of steel tubular piles. The load transfer path below seabed level is as follows: loads from reinforced concrete infill are transferred to steel tubular piles through frictional forces between reinforced concrete infill and steel piles. Therefore, mobilization of frictional forces between reinforced concrete infill and steel piles is essential to ensure that the piling system functions properly.

Helical stirrups are commonly used in the marine piling system of steel tubular piles with reinforced concrete infill. In fact, the use of links for column design in Britain is very popular. However, in the United States engineers tend to use helical reinforcement instead of normal links because helical reinforcement has the potential advantage of protection of piles against seismic loads. Moreover, when the piles reach the failure state, the concrete outside hoops cracks and falls off firstly, followed by the eventual failure of the whole columns. The peeling off of concrete outside helical reinforcement provides a warning signal before the sudden failure of piles. In addition, it can take up a higher working load than normal link reinforcement.

Soil Plug

In marine structures where piles are constantly subject to significant lateral and uplift forces induced by berthing operation and wave action, it is necessary to drive the piles to much greater depths. To avoid premature refusal so that insufficient soil cover may develop which is incapable of providing the required lateral and uplift resistance, tubular piles are normally driven open-ended so that they are driving to greater depths than piles with closed ends.

During initial driving process, open-ended steel piles are driven through the soils at their bases. However, shaft friction will gradually develop between the steel piles and soils inside piles at some time after pile driving. The hitting action of driving hammers induces forces to the soil and later it comes to a stage when the inertial forces of inside soils, together with the internal fictional forces exceeds the bearing capacity of soils at pile toes. Consequently, the soil plug formed is brought down by the piles.

It is practically possible to excavate all soils inside steel tubular piles and replace them completely by reinforced concrete. However, as engineers strive to produce economical design the extra cost associated with excavation of soil plug and filling of concrete could be saved in case the soil plug remains in position. Moreover, from the technical point of view it is considered unnecessary to remove the soil plugs because it serves to provide a platform for the placing of on-top infill concrete on one hand and to fill the void space below the infill concrete on the other hand. In addition, the soil plug is considered to be sufficiently compacted by pile driving action and is deemed to be stable during the design life of the piling system.

Problems in Design Aspect with the Marine Piling System of Steel Tubular Piles with Reinforced Concrete Infill

Radial Shrinkage of Reinforced Concrete Infill

Substantial amount of radial shrinkage and contraction occurs after concreting of concrete infill. This will hinder the load transfer from the concrete infill to steel piles because the bond may be ruptured by radial shrinkage resulting from the reduction of concrete volume. It is doubtful if frictional forces can be properly developed in this situation and therefore the load transfer mechanism assumed in design would no longer be valid.

To solve this problem, the first method is to install shear keys at regular spacing inside steel piles to ensure their rigid connection with concrete infill. However, for small diameter tubular piles (i.e. less than 800mm) where man-entry into the void of tubular piles is impossible, welding of shear keys on site is impracticable. In this connection, the tubular piles require special treatment and fabrication in fabrication yards which in turn increases the cost of construction.

Alternatively, expanding agents may be adopted in concrete mixes to ensure that there is no shrinkage after the concreting process. As such, the design assumption of load transfer between steel tubular piles and infill concrete by frictional resistance can be justified. However, reliance cannot be placed solely on the claim by the product manufacturers of the expanding agents' ability to swell. Intensive test on the expanding properties of expanding agents should be carried out on site to verify its actual performance.

Inadequate Pile Founding level

The most severe load on piers generally is the horizontal load due to berthing and mooring of large vessels. The design of piers is taken as an example to illustrate the importance of adequate pile founding level. Since the widths of open berth piers are relatively small so that they provide a short lever arm to counteract the moment induced by berthing loads. Moreover, the dead load of open berth piers are normally quite light and therefore the resisting moment provided by the dead load of pier structures may not be sufficient to counteract the moment generated by berthing loads. To aid in providing adequate resistance to the overturning moment by the berthing load, the soil resistance above bedrock contributes to the stabilizing moment. For commonly adopted marine piling type, i.e. driven steel tubular piles with reinforced concrete infill, driven piles can at most be founded on top of rockhead surface. In case the rockhead level is shallow (e.g. near shoreline), the little soil cover may result in inadequate lateral resistance to the berthing load.

Sometimes in contract document, minimum toe level is specified to provide sufficient length of soils for lateral and uplift resistance. Moreover, during detailed design ground investigation is usually conducted and the approximate level of rockhead is known. Therefore, to avoid the marine piles to be founded prematurely on boulders, minimum toe levels of marine driven piles are specified in contract. However, in case shallow rockhead is encountered for marine piles, the embedded length of steel tubular piles below seabed can hardly be extended beyond rockhead level by pile driving. Consequently, steel tubular piles may experience twisting and undergo deformation if excessive driving is applied to the piles in an attempt to drive through rock stratum.

To address the uplifting problems commonly encountered for marine piles, rock anchors which may consist of a high tensile bar or a stranded cable are provided for tension piles when there are insufficient soil covers to develop the required uplifting resistance. Hence, they are employed primarily for resisting uplift forces in the marine piling system of steel tubular piles with reinforced concrete infill. However, it can hardly withstand the lateral loads arising from horizontal loads like berthing loads and mooring loads. On the contrary, rock socket could be constructed for bored piles which serves the following three main purposes (Table 2):

(i) Rock socket friction and end bearing to resist vertical load;
(ii) Passive resistance of rock sockets contribute to resistance of lateral load; and
(iii) Socket shaft friction is also used for resisting uplifting forces. But only 70% of this capacity should be used because of the effect of negative Poisson ratio.

Hence, the use of bored piles together with rock socket could effectively solve the uplifting and lateral resistance problem associated with the marine piling system of steel tubular piles with reinforced concrete infill.

	Rock Socket	Rock Anchor
Vertical Loads	Yes	No
Lateral Loads	Yes	No
Uplifting	Yes	Yes

Table 2 – Comparison of functions between rock socket and rock anchor

Problems in Construction Aspect with the Marine Piling System of Steel Tubular Piles with Reinforced Concrete Infill

Construction Time

Subject to set and minimum toe level requirements, steel tubular piles are usually required under the contract to toe-in to Grade II or Grade III rockhead. Owing to the high strength of rock, it is considered as time-consuming to drive steel tubular piles from Grade V rock to Grade II or Grade III rock. For instance, the typical rate of penetration of steel piles by pile driving in Grade IV rock is about 0.4m/hr while the rate of penetration of steel piles in Grade III rock is about 0.2m/hr. Based on these figures, it is obvious that substantial amount of time is wasted for dealing with obstruction. However, in case bored piles are adopted to be installed at the same founding level, the rate of boring into bedrock should be much higher than the rate of driving into bedrock.

Obstructions Encountered During Pile Driving

In pile driving operation, proper selection of piling hammers is essential to prevent the damage of piles. For instance, a light hammer with higher drop causes a higher impact stress than a heavy hammer with lower drop provided that they generate the same energy per blow.

The seabed geology varies significantly from place to place and it is considered as not cost-effective to carry out borehole drilling at all pile locations. With limited information of the sub-soil condition, there is a risk of premature termination of piles owing to the presence of bounders and obstructions. It is not uncommon that bounders are frequently encountered during the driving of steel tubular piles. During driving, the piles are continuously subject to considerable reflected tensile stresses and compressive stresses. In case the pile sections are incorrectly aligned, the lack of straightness may induce significant bending stresses being locked in piles during pile driving. Moreover, if obstructions are encountered during pile driving, bending stresses would be induced in piles. Consequently, the pile head of the piles may be damaged during the piling driving into a boulder and extra time has to be spent on handling underground obstructions, which is deemed to be easily coped with in case bored piles are installed instead. For bored piles, the bounders could be smashed by free-falling a hammer to crush them into smaller pieces. Hence, the difficulty in handling underground obstruction during the construction of the marine piling system of steel tubular piles with reinforced concrete infill poses a potential problem for the piling system.

False Set in Pile Driving

For pile driving in certain soils like dense silt and weathered rock, the occurrence of false set phenomenon is not uncommon. During the driving process, negative pore water pressure is developed and the driven piles appear to have sufficient capacity during pile driving as the built-up of negative pore water pressure leads to a apparent temporary increase in driving resistance and strength. However, some time after the pile driving, the dissipation of this pore water pressure would render the piles insufficient bearing strength to resist the design loads. Sometimes, the presence of cracks along pile sections may bring about the problem of false set by the dampening effect of stress waves by these cracks. To avoid the problem of false set, a certain percentage of constructed piles should be selected to perform re-driving to check for the false set phenomenon.

Environmental Impact

For the construction of marine piles for piers and jetties which are close to the shoreline, the noise generated from pile driving operation may not be acceptable to a built-up environment, especially for schools, residential area and hospitals where quiet environment should be maintained. Therefore, the use of driving method in the marine piling system of steel tubular piles with reinforced concrete infill may prove unsuitable in urban areas.

Other than noise problem, the marine piling system also suffers from the demerit of excessive vibration generated during pile driving. In particular, the situation becomes worse when there are some sensitive structures, utilities or machinery nearby. Since pile driving involves ground displacement, the effect of ground movement, together with vibration effect of driving operation, may cause damage to nearby structures and utilities nearby.

Conclusion

This paper presents the different elements of the marine piling system of steel tubular piles with reinforced concrete infill and attempts to address the technical problems and demerits in employing the use of the system. The conclusions of the paper are as follows:

(1) From the design point of view, the marine piling system of steel tubular piles with reinforced concrete infill exists a potential design flaw of the occurrence of radial shrinkage leading to the failure in effecting the transfer of forces between the reinforced concrete infill and the steel tubular piles. Moreover, in locations with shallow rockhead it may be difficult to mobilize sufficient lateral resistance against loads like berthing loads owing to the limitation of the capability of pile driving into rockhead.

(2) From the construction point of view, the construction time required to deal with obstruction is higher when compared with other available options like bored piles. Moreover, difficulties arise during pile driving when boulders and obstructions are come across and special attention has to be made related to the problem of false set during pile driving. Other than the consideration of technical aspects of the piling system, it poses various environmental problems like noise, vibration and lateral movement arising from pile driving operation.

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Chapter 5. Piles and Foundation

1. What is the significance of reinforced concrete infill in marine piling system of steel tubular pile with reinforced concrete infill?

Reinforced concrete is designed to fill the void space inside the steel tubular piles from pile cap to a certain distance below seabed. As mentioned earlier, steel tubular piles above seabed level is assumed in design to be completely corroded when approaching the end of design life. As such, loads from pile caps are transferred directly to reinforced concrete infill instead of steel tubular piles. The load transfer path below seabed level is as follows: loads from reinforced concrete infill are transferred to steel tubular piles through frictional forces between reinforced concrete infill and steel piles. Therefore, mobilization of frictional forces between reinforced concrete infill and steel piles is essential to ensure that the piling system functions properly.

2. Why "inadequate pile founding level" commonly occurs in piles of piers?

The most severe load on piers generally is the horizontal load due to berthing and mooring of large vessels. The design of piers is taken as an example to illustrate the importance of adequate pile founding level. Since the widths of open berth piers are relatively small so that they provide a short lever arm to counteract the moment induced by berthing loads. Moreover, the dead load of open berth piers are normally quite light and therefore the resisting moment provided by the dead load of pier structures may not be sufficient to counteract the moment generated by berthing loads. To aid in providing adequate resistance to the overturning moment by the berthing load, the soil resistance above bedrock contributes to the stabilizing moment. For commonly adopted marine piling type, i.e. driven steel tubular piles with reinforced concrete infill, driven piles can at most be founded on top of rockhead surface. In case the rockhead level is shallow (e.g. near shoreline), the little soil cover may result in inadequate lateral resistance to the berthing load.

3. Which type of pile cap transfers loads equally to piles, flexible pile cap or rigid pile cap?

Loads from columns transferring to pile cap induce tensile forces at the bottom of the cap. For instance, by using truss analogy to analyze a pile cap sitting on two piles with a column at the centre of the pile cap, the tensile force at the bottom is proportional to the pile spacing and is inversely proportional to depth of pile cap. The bottom reinforcement is designed to resist the tensile stressed generated from loads in columns. Sometimes, reinforcement may be designed at the top of pile caps which serve as compression reinforcement. This type of reinforcement is required in case there is a limitation on the depth of pile caps. Similarly shear reinforcement is introduced to the pile caps in case there is a restriction to the depth of pile caps.

Consider that loads are applied at the centre of a pile cap.

For the case of rigid pile cap, owing to the effect of interaction of individual piles, the central piles tend to settle more than the edge piles when the pile cap is under loading condition. For the pile cap to be rigid, the local deformation of central piles would not occur. Instead, the stiff pile cap would transfer the loads from the central piles and redistribute them to the outer piles. Therefore, piles at the edge take up a higher fraction of the total loads and are subjected to higher axial and bending loads in case the pile cap is stiff. In the extreme case, the side piles may take up as much as about two to three times the loads in the central piles and this may lead to the failure of these edge piles.

For flexible pile cap, load taken up by individual piles are different because the deformation of pile cap enhances non-uniform distribution of loads among piles. The piles closer to the load tend to share more loads when compared with those which are located far away from the loads. The difference of loads induced in piles increase with the flexibility of pile cap.

4. What are the functions of different reinforcement in a typical pile cap?

Loads from columns transferring to pile cap induce tensile forces at the bottom of the cap. For instance, by using truss analogy to analyze a pile cap sitting on two piles with a column at the centre of the pile cap, the tensile force at the bottom is proportional to the pile spacing and is inversely proportional to depth of pile cap. The bottom reinforcement is designed to resist the tensile stressed generated from loads in columns.

Side reinforcement may not be necessary in pile cap (L.A. Clark (1983)). In fact, the primary aim of the side reinforcement is to control cracking. However, as most pile caps are hidden from view and it is considered not necessary to provide side reinforcement to pile caps based on aesthetic reason.

Sometimes, reinforcement may be designed at the top of pile caps which serve as compression reinforcement. This type of reinforcement is required in case there is a limitation on the depth of pile caps. Similarly shear reinforcement is introduced to the pile caps in case there is a restriction to the depth of pile caps.

L. A. Clark (1983) *Concrete Bridge Design to BS5400* Construction Press, Longman Group Limited pp.94

5. Can both Pile Drive Analyser (PDA) and Pile Integrity Test (PIT) be used for checking pile capacity?

Pile Drive Analyser is a high-strain dynamic test to determine the force and velocity response of a pile to an impact force applied axially by a driving hammer at the pile top. It is applicable to driven piles or small diameter bored piles. The operation measures the elastic deformation of a pile after a hammer blow and is mainly used to check the ultimate capacity of piles. However, it may also be adopted to detect damages in pile body and obtain the friction profile along the pile shaft.

Pile Integrity Test is a low-strain dynamic test which involves the use of a small vibrator or a light hammer. It is applicable to small diameter driven concrete piles and large diameter bored piles. It can be used to check the following properties:

(i) Quality of concrete (e.g. honeycombing)

(ii)Location and type of damages
(iii) Estimation of pile length

However, it is mainly used to check the integrity of piles only and it may be used to deduce the pile capacity.

6. Should bentonite be added to improve the stability of grout?

For unstable grout, particles will come out of the grout suspension leading to incomplete grouting and clogging of pipes. The stability of grout can be improved by adding additives such as bentonite. However, bentonite should not be used with very fine cements because its grain size is bigger than that of fine cements. Tests conducted previously confirm that a grout with bentonite is less stable under pressure.

It is commonly accepted that a fissure may be penetrated by grout with the grain size about 3-5 times smaller than the aperture of fissure. Hence, OPC cement may penetrate fissures of aperture greater than 0.4mm while microfine cement and ultrafine cement may penetrate fissures of aperture greater than 0.1mm and 0.03mm respectively.

7. How can marble cavities and karstic features be detected in ground investigation?

Marble is metamorphic rock derived from limestone and is dissolves in slightly acidic water to form cavities (partly filled with debris). It poses great problem for construction of tall buildings which requires the seating of firm foundation.

One of the way to identify marble cavities and karstic features is to employ a combination of rotary drilling and micro-gravity method. Micro-gravity method involves the measurement of minute variations in gravitational pull of the Earth and interpretation of the presence of cavities from them. The principle of the technique is to locate areas of contrasting density in the sub-surface. As a cavity represents a lower density when compared with its surrounding soils, the subsequent small reduction in the pull of the Earth's gravity is observed over the cavity.

8. Is Hiley's formula suitable for estimating capacities all driven H-piles?

In Hong Kong, the local practice of driven H-piles is to adopt dynamic formula like Hiley's Formula to drive piles to set and verify its capacity by static load test for a certain proportion of these piles.

However, in long piles and in some special ground conditions, the capacity of piles predicted by Hiley's Formula is not on a conservative side. To address these problems, static load tests can be carried out to determine shaft resistance of H-piles and these data is used for designing the pile lengths. When the piles are actually driven into ground, it capacity can then be checked by adopting dynamic load testing such as PDA test. Alternatively, wave equation analysis can be used for estimating the soils resistance of the ground and the pile penetration required.

9. What are the potential problematic areas in bell-out of bored piles?

Owing to the relatively low presumed bearing value for founding rock (i.e. 5MPa for Grade II rock with total core recovery of 85% and unconfined compressive strength more than 25MPa), it leads to the necessity to form bell-out in order to spread the pile loads in larger area, thus reducing the bearing stress at pile toes.

Firstly, in Hong Kong most of rock belongs to igneous rocks whose unconfined compressive strength is generally higher than normal concrete (no reinforcement in concrete in bell-out region). Therefore, someone may query the reason of replacement of strong rock by weaker concrete during the formation of bell-out.

When piles are concreted by tremie concrete method, soil sediments at the pile toe inside the bell-out are pushed aside and trapped in the tip of bell-out. Consequently, this would impair the functionality of the bored piles.

To maintain the stability of hanging side slope of bell-out, it requires the bell-out to be formed wholly inside sound rock. However, the rock above bell-out may be weather rock so that the hanging rock is liable to fall down and soil may collapse.

10. Is it totally unacceptable that soil sediments are found at pile toe of bored piles?

The pile toe is usually cleared by airlifting prior to concreting. However, once the air-lifting operation ceases and concreting operation are not carried out simultaneously, suspended sediments would tend to settle and form the soil sediments at pile toe of bored piles. The layer of soil sediments is considered to be unacceptable by most engineers as they claim that it would impair the structural performance of bored piles.

For bored files with bell-out, the rock socket provided during the formation of bell-out tends to provide confinement to soil sediments. The horizontal stress will increase with the applied stress. Hence the settlement of soil sediments is very small and would not affect the performance of bored piles.

11. In some pile design, the settlement of piles are not checked. Is it correct?

The performance of piles mainly consists of the two elements, namely ultimate bearing capacity and settlement. The local practice of pile design is place emphasis on checking if the bearing capacity of piles would be exceeded.

Engineers tend to adopt the approach that bored piles are designed to be founded on bedrock while for driven piles they are driven to very stiff stratum with SPT N values greater than 200. Owing to rigid and firm foundation on which the piles are seated, it is therefore assumed that the amount of settlement shall be limited. On the other hand, there is practical difficulty in assessing how much settlements are considered acceptable owing to limited available data.

12. Why can't normal Reversed Circulation Drills function in shallow rock conditions?

Reversed Circulation Drill (RCD) is normally used for forming large diameter rock socket. The method involves the exertion of a download force of roller cutter bits on rock, together with the action of rotation and grinding of bits on rock. The cuttings are then removed by reverse circulation. The water and cuttings are airlifted through a central drill pipe, which is also used for rotating the drill bits.

To facilitate the grinding action on the rock, about 15 tons of force is used for each cutter. With such a high bit force, the drill frame has to be stationed by attaching to pile casing of bored piles. Therefore, during the drilling operation, the pile casing is prevented from lifting up by the weight of drill rig and pile casing and the frictional forces developed between the ground and pile casing. Hence, in shallow rock conditions with short length of pile casings, it may affect the stability of RCD drill rig.

13. Can down the hole hammer function below water table?

Down the hole hammer has been used extensively to form pre-bored holes as rock sockets for mini piles and pre-bored H piles. The hammer functions by driving repeatedly a drill bit using compressed air on the rock. However, the use of down the hole hammer is normally limited to hole diameter of 600mm.

In using down the hole hammer, compressed air serves to drive the drill bit and to expel the cuttings which are blown out to the air at ground level. However, for driving the hammer about 30m below ground water level, the air pressure has difficulty in coping with great water pressure. Moreover, blowing of cuttings by compressed air also dewaters the nearby soils. As a result, settlement of nearby ground may occur which is undesirable.

14. What is the purpose of shaft grouting of deep foundations?

In shaft grouting operation, tube-a-manchette pipes are fixed at regular spacing to the reinforcement cage. After concreting barrettes/bored piles, a small volume of water is injected under high pressure into these pipes to crack the concrete. The cracking process should be carried out within 24 hours after concreting. The purpose of cracking is to create a path for grout to go through. About a week after concreting of barrette, grouting is then carried out in these pipes to improve the friction between the foundation and the surrounding soils.

15. What are the meanings of the mathematical terms in failure criteria of pile load test?

Load tests are conducted to verify the design assumptions and parameters such as pile friction in soils and sock socket capacity. There are various failure criteria in current construction industry to determine ultimate load resistance of piles in pile load test. For instance in 90% criterion of Brinch Hansen, it is based on the laboratory measured stress-strain relations of soils and a point is identified in which soil fails. This essentially aims at looking for the ultimate bearing capacity and hence the ultimate loads. In fact, this is not intended originally for piles.

For some failure criteria, it does not target at finding out the ultimate pile capacity. Instead, it looks for the process for onset of soil yielding at the base of pile toe an allows for controlled displacement. For example, one of these criteria is reproduced as follows:

Criterion for maximum movement = $(PL/AE + d/120 + 4)$ mm

where P is the load, L is pile length, A is the area of pile and d is pile diameter.

This failure criterion was developed based on small diameter driven piles. The term PL/AE refers to elastic shortening of piles. For end-bearing piles, this term is acceptable fro usage. However, for friction piles this may not truly simulate the actual shortenings of piles because frictional forces along the pile also come into play. The term $(d/120 + 4)$ represents the amount of soil movement which triggers the yielding of soil beneath pile toe.

16. Why is sleeving applied in piles constructed on slopes?

For high-rise buildings constructed on steep cut slopes, these buildings are usually supported by large diameter piles. Though the piles are founded at some depth below the slopes, lateral load arising from wind on buildings may induce loads on the slop and causes slope failures. For shallow depths of slope which is marginally stable, it is more vulnerable to slope failure.

Hence, an annulus of compressible material called sleeving is introduced in piles so as to reduce the transfer of lateral loads from buildings to slopes.

17. In the installation of strain gages in driven H-piles to measure loads, why should they be normally used in pairs?

Strain gages are often installed in driven piles to measure the load distribution along the piles. They have to be protected from being removed as the pile is driven into the ground. Protection of strain gages is achieved by welding channels or angles for enclosure of stain gages.

Strain gages should always be installed in pairs located back to back on the same piece of steel. For instance they may be placed back to back on either side of web of H-piles. Only one gage mounted on the cross section of H-pile is not too useful because it may be affected by an unknown degree by bending moment. Hence, the results of axial load may appear to be doubtful.

18. Is Hiley's formula suitable for clayey soils?

The basis of all dynamic formulae is the conservation of energy in which energy applied to the piling system by driving hammer is equal to the energy required to penetrate the soils with energy losses during the process. The Hiley's formula is the most popular dynamic formula used in Hong Kong.

The Hiley's formula is developed for conditions of soils consisting of sands and gravels. Under this soil condition, it is assumed that there is no time dependency between dynamic and static resistance of piles. However, for clayey soils the static strength development varies greatly with time and the load capacity of driven piles in clayey soils may be lower than that of the instantaneous ones.

19. Which of the following stages is noisier, at the ending of pile driving operation or at the end of pile driving operation?

When the piles are progressively driven into the ground, the pile section above the ground declines. As a result, the degree of damping on the piles increases. Moreover, the area of exposure of piling surface reduces, thereby reducing the area generating noise form piles. Hence towards the end of pile driving operation, the noise level shall be reduced accordingly.

Noise screen shall be installed to tackle the noise problem. Noise screen made of plywood might not be sufficient because it tends to reflect the noise back to the site and increase the reverberation of the site. Instead for the face of noise screen facing the piling operation shall be lined with a layer of sound-absorbing material such as glass fibre. Moreover, openings on noise screen should be avoided because it can substantially reduce the performance of noise screen.

Chapter 6. Roadworks

1. Does the use of concrete road enhance fuel saving when compared with bituminous road?

Concrete road belongs to rigid pavement and they do not deflect under traffic loads. On the contrary, bituminous pavement deflects when subjected to vehicular load. As such, for concrete road no extra effort is paid on getting out of deflected ruts which is commonly encountered for bituminous pavement. Hence, vehicles using concrete road use less energy and there is about 15-20% less fuel consumed when using concrete road when compared with bituminous road.

2. Is vehicle parapet strong enough to contain vehicles?

The majority of local parapets are 1.1m high and they are designed to resist impact from a 1.5ton car moving at a speed of 113km/hr. In some locations such as in the vicinity of railway lines, barriers with 1.5m high are provided to contain a vehicle with 24ton at a speed of 50km/hr.

The impact situation for vehicles varies from event to event and they are dependent on the speed, size and angle of incidence of the impacting vehicle. Though full-scale crash test is the simplest way to prove their performance, computer simulation has been used extensively owing to its lower in cost. Based on the results of computer simulation and crash tests, it is established that the said parapets comply with international standard for safe usage.

3. In backcalculation of moduli in Falling Weight Deflectometer, why are non-unique solutions generated?

Falling Weight Deflectometer is a non-destructive test applied to pavement for structural evaluation. The loads applied in Falling Weight Deflectometer are of impulse type (quasi-static load in Benkelman Beam) and it serves to simulate actual truck wheel load.

Surface deflections induced by the impulse load are measured by some sensors located at the pavement and the pavement surface's deflections form a deflection basin. The measured deflections can then be adopted to estimate the elastic moduli of pavement structural layers by backcalculation.

In backcalculation process, a pavement model is firstly established to represent the exiting pavement structure. Trial values of moduli of structural layers are used to calculate the theoretical deflections of pavement model and these values are compared with the measured deflections in Falling Weight Deflectometer. The trial moduli are then adjusted iteratively until both values agree closely with one another.

However, it is observed that the backcalculated moduli based on the analysis of Falling Weight Deflectometer is dependent on software and user. Different computer programmes can generate different values of backcalculated moduli from the same deflection basin. In fact, for a given deflection basin, there are numerous combinations of trial modulus which can satisfy the deflection envelope with a certain accuracy level and hence it may result in non-unique solution developed.

4. What are the differences between capping layer and sub-base?

For weak and poor quality subgrade, there is a need to increase the thickness of pavement to compensate for it. In order to save cost for sub-base which is relatively expensive, the concept of capping layer is introduced in which capping materials of cheap but strong nature are used to cap the weak subgrade. In this way, the thickness of expensive subgrade is not required to be increased.

Crushed gravels and rockfill may be suitable options of capping material. In essence, capping material should be readily available at low cost. The capping layer not only serves to strengthen the subgrade, but also protect the road formation during construction. It serves as haul road for construction traffic during construction stage. Moreover, it protects the subgrade from weathering such as wetting.

5. Is California Bearing Ratio method suitable for pavement design?

California Bearing Ratio method is essentially a test for bearing capacity of ground under an application of load at a low rate of penetration. In actual pavement, it is the dynamic stiffness of pavement which is of paramount importance because the pavement is subjected to repeated loading at low stress levels. Design procedures based on dynamic stiffness can be adopted but difficulties are encountered in selecting appropriate modulus for dynamic stiffness.

6. What is the difference between sag gully and on-grade gully?

A sag gully is a gully installed at a low point in roads and stormwater would pond up the gully. An on-grade gully is a gully installed in a sloping road where any excess flows may bypass the inlet and flow to another one downstream. Generally speaking, the entry capacity of sag gully can be estimated from hydraulic design. But for on-grade gully, owing to complicated hydraulic behaviour, the entry capacity can hardly be predicted in accuracy by mere calculation.

Chapter 7. Slopes

1. Does liquefaction occur to sand only?

In liquefaction, the pore water pressure builds up steadily and eventually approaches a value equal to the confining pressure. In an earthquake, however, there is not enough time for the water in the pores of the soil to be squeezed out. Instead, the water is trapped and this avoids the soil particles from moving closer together. Consequently, this results in an increase in water pressure which reduces the contact forces between the individual soil particles, thereby softening and weakening the soil. Eventually, soils particles lose contact with each other and behave like a liquid.

Hence, the type of soils which is susceptible to liquefaction is the one like sand whose resistance to deformation is mobilized by frictional forces between particles under confining pressure. In case the soil is fine grained, cohesive forces tends to develop between these fine particles and it is difficult to separate them. Therefore, sand with increasing content of fines tends to increase its resistance to liquefaction.

The consequence of liquefaction is that the subsequent settlements after liquefaction may damage the overlying structures. Moreover, for sloping ground lateral flow may result which is undesirable. Liquefaction only occurs to saturated soils.

2. What are the measures to reduce the effects of soil liquefaction?

To reduce the effect of soil liquefaction, it is intended to reduce the pore water pressure induced during earthquake shaking. This can be achieved by providing better drainage in soils (e.g. wick drains, sand drains etc.) and densification of soils (e.g. vibroflotation, dynamic compaction etc.).

Liquefaction hazards can be reduced by improving the drainage ability of the soil. If the pore water within the soil can drain freely, the build-up of excess pore water pressure would be reduced accordingly.

3. What is the difference in failure slip surface between slopes with cohesive and granular materials?

When cohesive strength is zero (i.e. slopes of granular types), the slip surface is of shallow failure type and is parallel to the slope surface.

When friction angle is zero (i.e. slopes of clayey types), the slip surface is if deep seated failure. The factor of safety of slopes is nearly independent of the angle of slopes because the weight of deep seated failure regime is much greater than the slope.

Normally, non-circular failure surface is always more critical than circular one for two dimensional analysis.

4. Does cutting slope cause slope deformation or slope failure?

Slope cutting causes stress relief in slopes which may cause slope movement. For instance, for weathered rocks the horizontal stresses would be relatively low when compared with normally consolidated soils. Consequently, a major cut on the slope formed by weather rock may result in the development of tensile stresses in the slope, leading to slope movement.

5. Why do landslides occur though the rainfall has not led to full saturation in the sliding zone?

From soil mechanics, it tells us that unsaturated soils get its strength from three main components, namely, friction, cohesion and suction. In building a sand castle in a beach, experience tells us that when sand is too dry or too wet, the castle can hardly be built. However, when the sand is partially saturated, the suction (negative pore water pressure) holds the sand together and provides the strength the build the castle.

In the event of intensive rainfall, the soils cannot get away the water at the rate it is penetrating into the slope and this results in wetting up of the subsurface soils. When the slopes gets too wet (but not yet saturated), it loose much strength in terms of suction (negative pore water pressure) and results in slope failure. This occurs despite the fact that the sliding mass is well above the ground water table.

In Hong Kong about 80% of landslides occur owing to erosion and loss in suction. Only less than 20% of landslides occur as a result of increase of pore water pressure, leading to the decrease in shear strength.

6. What are the major causes of rockfall?

The causes of rockfall can be broadly classified into the following two reasons:

(i) Freezing and thawing and the subsequent development of vegetation root pressure in slopes is one of the major causes of rockfall in some countries like Europe. Moreover, rockfall can also be triggered by heavy rainstorms which bring about surface erosion and generate water pressure in rock joints.
(ii) Rockfall can also be induced by poor joint patterns and low strength and water pressure in joints.

7. Why does flexible wire rope nets effective in stopping rockfall?

A rockfall barrier has to serve basically the following two purposes:

(i) It has to be sufficiently high so that it would not be jumped over by rock bounders.
(ii) It has to resist the impact of rockfall without structural failure.

For rockfall energy greater than 2,500kJ, it requires the installation of earth dams, underpinning (stabilized) or even complete removal. For rockfall energy less than 2,500kJ, it can be resisted by rigid structures made of concrete or steel. However, the recent trend is that most of these rigid structures are replaced by flexible structures like flexible wire rope nets. For total work on barrier system, it is defined as force times displacement. The merit of flexible system lies in the small forces generated with large displacement when compared with larges forces with little displacement in rigid structures. As a result, the flexible structures can be designed with low impact forces leading to better and economical design.

8. Will the posts of rockfall barrier be damaged when it is hit directly by rock boulders?

Rockfall barrier is intended to absorb the energy of rockfall safety without the need of future regular maintenance. The net of rockfall barrier is best to be arranged like Olympic Rings with interlacing (called ringnet) and it has proved to be very effective in its energy absorption capacity.

The main function of post of rockfall barrier is to provide support to the net. Impacts to posts should not result in the collapse of the structure. In the event of directly impact of rocks with the posts, there is a break devise at the post's base plate which is designed to allow the post from separating the base plate. In this way, it serves to protect the anchor of base plate from being damaged.

9. How do soil nails help to improve the stability of slopes?

It is commonly believed that with the introduction of soil nails to slopes, this new combination of elements possesses higher shear strength than the original soils. In the context of Rankine's active state, soil nailing serves to provide horizontal restraint to guard against active failure.

Moreover, when the soils inside the zone of failure are improved, block failure based on Coulomb is still feasible. By installation of soil nails, it helps to defer the original failure planes of slopes to a greater depth inside the slope, which it is normally of high stability condition with less ingress of surface rainwater.

10. What are the rationales of Observational Method in

geotechnical works?

The idea of Observational Method was first discussed by Peck in the Rankine Lecture in 1969. The Observational Method is commonly adopted in geotechnical works in construction phase, though it is also feasible in design stage.

In essence, in the conforming design by engineers during planning stage, the design is usually based on over-conservative approach or most unfavourable conditions owing to a lack of precise and actual site information. During subsequent construction, with precise site information and condition available the Observational Method is adopted in which the original design is revised based on most probable conditions with instrumentation monitoring. If the monitoring results show that performance of the revised design approaches the limit of acceptable level of risk, then it shall be reverted to planned modification which is based on most unfavourable conditions and hence the level of risk is lowered back to the original design. Otherwise, the revised design shall continue and this results in cost reduction without comprising safety of works.

However, care should be taken in implementing Observational Method when rapid deterioration of the site may occur so that there is insufficient time for carrying out the planned modification. For instance, rapid deterioration can result from development of high pore water pressure in heavy rainfall or burst watermain.

11. How does reinforcement function in embankment built on soft clay?

The factor of safety for the embankment constructing on soft clay is minimum at the end of construction. After that consolidation takes place, thereby increasing the shear strength of foundation soils. Reinforcement is normally introduced to maintain the stability owing to the following reasons:

(i) The reinforcement at the clay surface, which is capable of carrying tensile forces, generates shear stresses to resist the lateral deformation of clay and improves its bearing capacity.

(ii) The reinforcement could also hold in equilibrium the lateral trust developed by the fill above so that it reduces the stresses which tends to cause failure of clay foundation.
(iii)The reinforcement has a tendency to drive the failure mechanism deeper in the soft clay, which should possess higher shear strength because its strength generally increases with depth.

12. How can decomposed dolerite dykes affect slope stability?

Decomposed dolerite dykes contain high clay content display high plasticity. Moreover, the material is generally of low permeability than decomposed granite which is found in many slopes.

The presence of decomposed dolerite dykes in a direction parallel to slope surface may cause slope failure after heavy and prolonged rainfall. Owing to the low permeability of decomposed dolerite dykes, perched water table would be developed above the dykes and this essentially increase the positive pore water pressure which reduces the shear strength of soils.

The presence of decomposed dolerite dykes in a direction perpendicular to slope surface may also cause slope failure after severe rainstorm. The low permeability of the material tends to act as a dam building up the local groundwater level for water coming down from upper slopes. The increase of water level behind the "dam" causes an increase in pore water pressure so as to reduce the strength of soils in slope, leading to subsequent failure.

13. What are the potential problems of using clayey backfill in reinforced fill?

Reinforced fill consists of reinforcement embedded in fill with facing. With backfilling material of clayey nature, it may pose problems to potential corrosion of steel reinforcement. The low permeability of cohesive fill materials tends increase the duration of contact between reinforcement and water and may cause corrosion problems. Moreover, consideration should be given if cohesive materials could also achieve the required compaction.

14. How does geogrid function in reinforced fill?

Geogrid allows the fill on one side of the grid can key with fill on the other side of the grid. Hence, it is different in its interlocking ability when compared with strip soil reinforcement. The keying-in of both sides of fill could be achieved by compaction and static load above the fill.

To trigger shearing across the plane of geogrid, work has to be done to dilate the soils and overcome the frictional forces.

15. What are the functions of drainage system and protective covers of slopes?

In Hong Kong the angle of fill slope is about 30^o – 40^o to the horizontal while the angle of cut slope is about 50^o – 60^o to the horizontal. To protect the slope from surface erosion and water infiltration, it is covered with impermeable hard cover like chunam and shotcrete. Chunam is a mixture of soil, cement and lime and is usually applied in two layers with the thickness of each layer of 25mm. For shotcrete, mesh reinforcement may be provided inside it to enhance tensile strength of cover, thereby reducing the risk of tensile cracking of slope cover.

Weepholes are normally provided in slopes to prevent building up of water pressure in the slope and subsequently this causes cracking and disintegration of slope cover. For gentle slope, hydroseeding may be used and geofabric may be introduced on slope surface to guard against possible surface erosion.

U-channels are provided on the crest and toe of slope to divert and collect the rain falling on slope surface. Catchpits are provided at the intersection or junctions of drains to avoid possible splashing of water.

16. Why are filled slopes vulnerable to slope failure?

Filled slopes constructed in many decades ago are mostly sub-standard.

The relative density of filled slopes may be below 85% and is readily subjected to liquefaction. To rectify the situation, the sloped are reconstructed by excavation of 3m measured vertically from slope surface. Then, compaction should be carried out in thin layers to achieve in-situ density of not less than 95% of maximum dry density. After compaction, the compacted layer would not vulnerable to liquefaction failure. Moreover, it is less permeable than loose fill upon compaction and prevents water entry into underlying soils inside the slope.

For the case of Hong Kong, most fill slopes constructed before 1977 were formed by end-tipping so that they are in a loose state and poses hazard to developments nearby.

17. What are the effects of pile groups on slope stability?

The ground conditions in hillsides of Hong Kong normally consist of colluvium overlying weathered and fresh rock. The groundwater flow in sloping ground may involve both perched and main water flow. As permeability of soil tends to decrease with depth, groundwater flow may take place in the upper aquifer which contains a perched water table.

The presence of pile groups below groundwater level hinders the flow of groundwater, leading to an increase on groundwater level. For major development in hillsides with plenty of deep foundations, the effect of rise in water table is sever. The increase in water table owing to damming effects of pile groups decreases the stability of slopes by reduction of shear strength of soils and may eventually cause slope failures.

The rise in groundwater levels tend to increase with slope angle and the depth of groundwater flow.

18. Does the presence of colluvium beneficial to slopes?

Colluvium refers to common surface deposit rolling down hillsides under the action of gravity. It differs from residual soils which, upon decomposition the discontinuity, joints and textures are preserved. The rolling action of colluvium destroys these features.

Colluvium displays some important features which deserves attention:

(i) It appears to be more resistant than residual soil to erosion and tends to offer protection to the soils beneath.
(ii) Its relatively lower permeability when compared with underlying residual soils may form a perched water table at its base during heavy rainstorm.
(iii)For residual soils, when slope cutting is carried out the presence of relict joints may affect the slope's stability. However, the base layer of colluvium is also a plane of potential failure location.

19. What are the limitations in multi-stage test of triaxial tests?

The maximum load acted on the soil specimen is limited because a highly deformed soil sample is not suitable for further testings. For instance, soft samples like clay display large failure strain and hence it may be not considered acceptable for multi-stage trial axial tests.

Moreover, multi-stage trial axial tests may not be suitable for residual coils whose cohesion is established based on the remaining rock strength mass. At the stage of shearing, part of cohesion may be destroyed and it is irrecoverable in other stages of triaxial tests.

20. How do horizontal drains help to stabilize slopes?

The use of horizontal drains to enhance slope stability in the following ways:

(i) It enhances an increase in soil strength by lowering the degree of saturation of soils.
(ii) It reduces the pore water pressure in the region of potential slip surface.
(iii)It hinders the development of seepage forces.

Backcover

About the Author

Vincent T. H. CHU (朱敦瀚) , famed as *walking encyclopedia of civil engineering* (有 Civil 百科全書的外號), obtained the degree of civil and structural engineering in the University of Hong Kong. He is the author of the monthly column "The Civil FAQ" in the Hong Kong Engineer published by the Hong Kong Institution of Engineers and is the author of the civil engineering monthly columns "The Civil Q&A" and "The Civil Corner" on the websites on World Federation of Engineering Organization and the University of Science and Technology (American Society of Civil Engineers – International Student Group) respectively. He is the recipient of the Ombudsman's Award 2007 under complaint-related category and Young Engineer of the Year Award 2008 (Merit) organized by the Hong Kong Institution of Engineers. He is also the author of the engineering book "200 Question and Answers on Practical Civil Engineering Works", which is widely publicized and posted on the websites of following engineering organizations and universities around the world:

EUROPE

Posted on Engineering Websites

- European Council of Civil Engineers ECCE
  http://www.ecceengineers.eu/papers/index.php

- Institution of Civil Engineer (United Kingdom) http://www.ice.org.uk/knowledge/document_details.asp?Docu_id=1715&intPage=4&faculty

- German Federation of Technical and Scientific Organisations DVT
  http://www.dvt-net.de/intern.html

- Slovak Chamber of Civil Engineers (斯洛伐克共和國)
  http://www.sksi.sk/buxus/generate_page.php?page_id=1

- Hemsley Orrell Partnership (Consulting Civil & Structural Engineers)
  http://www.hop.uk.com/information.html

- Imperial College London
  http://civeselib.wordpress.com/ (posted on 30 June 2008)

Distribution to Members

- Schweizerischer Ingenieur- und Architektenverein (SIA - Switzerland)

- The Federation of the Scientific - Engineering Unions in Bulgaria

ASIA

Posted on Engineering Websites

- Japan Society of Civil Engineers
  http://jsce.jp/index.pl?section=bookReview

- Turkish Chamber of Civil Engineers
  http://e-imo.imo.org.tr/Portal/Web/IMO.aspx?WebSayfaKey=815

- Japan Federation of Engineering Societies
  JFES-IAC E-News No. 5 (7/2008)

http://www.jfes.or.jp/activitie/iac_news/jfes-iac_e-news_005.pdf

- Philippine Institute of Civil Engineers
  http://www.pice.org.ph/console.htm

- Mongolian Association of Civil Engineers
  http://www.mace.org.mn/index.php

- The University of Science and Technology (American Society of Civil Engineers – International Student Group)
  http://ihome.ust.hk/~asce/

- The Alumni Newsletter of the University of Santo Tomas Civil Engineering Department (Philippines)
  http://lab6report.wordpress.com/2007/05/09/a-weblog-devoted-to-ust-civil-engineers/

Distribution to Members

- Institution of Engineers, Pakistan

- The Hong Kong Institute of Vocational Education (Morrison Hill)

- The University of Hong Kong (Civil Society)

- City University of Hong Kong

NORTH AMERICA

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- Deep Foundations Institute
  http://www.dfi.org/

- The CivilEngineer.org
  http://www.thecivilengineer.org/general_civil/library_general_civil.html

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- Structural Engineers Association of California (SEAOC)
- Arup –Washington DC Office

OCEANIA

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- Engineer Australia (Informit e-library)
  http://www.informit.com.au/elibrary_ieleng.html

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- Monash University (Australia)

AFRICA

Posted on Engineering Websites

- Institute of Professional Engineering Technologists (South Africa)
  http://www.ipet.co.za/news/OctFinalPDF2008.pdf

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- South African Institution of Civil Engineering

ISLANDS OR OTHERS

Posted on Engineering Websites

- World Federation of Engineering Organizations
  http://www.wfeo.org/

- The Barbados Association of Professional Engineers
  http://www.bape.org/

Distribution to Members

- World Council of Civil Engineers (WCCE)

The author has established a free Civil FAQ email service called "Ask Vincent Chu" (email: askvincentchu@yahoo.com.hk) in which he would answer civil engineering queries raised from engineers (especially young engineers).

Interested readers could refer to the personal interview of the author regarding his further background information:

(i) Face Magazine on 2 December 2008
http://education.atnext.com/index.php?fuseaction=Article.View&articleID=11925677&issueID=20081203

(ii) Jiu Jik 招職 on 30 September 2008
http://www.jiujik.com/jsarticle.php?lcid=HK.B5&artid=3000022089&arttype=LEISU&artsection=CAREER