# A Self-Learning Manual

# Mastering Different Fields of Civil Engineering Works (VC-Q&A Method)

*Vincent T. H. CHU (朱敦瀚)*

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# *Preface*

This Manual presents a systematic way to help engineers master different disciplines of civil engineering in a easy manner and within a short time. I feel proud to introduce the new method "VC-Q&A Method", which makes use of the easy-reading style of Q&A to achieve efficient and effective mastering of different fields of civil engineering.

Unlike other professionals, civil engineers are "deemed" to be equipped with a wide range of engineering knowledge. It is extremely rare that civil engineers are solely responsible for a particular field of engineering without touching on other disciplines. To put it simple, when one builds a structure, you not only have to understand its concrete nature itself, but also its foundation, its associated drainage and sewage infrastructure. Therefore, it is of utmost importance for civil engineers to appreciate and learn other disciplines of civil engineering other than their own expertise.

In fact, the idea of "VC-Q&A Method" originates from my past learning experience. My knowledge of different fields of civil engineering was acquired mainly through the curious questioning of prevailing civil engineering practice and subsequent tedious searching for answers. This mode of critical thinking and the essence of issues are embodied into this Manual. The essence of "VC-Q&A Method" is to let readers experience my previous thinking path through my Q&A and guide them to use Q&A approach to learn and study further. I wish that this Manual presents a big step forward in helping practicing engineers to learn different fields of civil engineering in the most interesting and easy way.

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

Vincent T. H. CHU
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# *1.* VC-Q&A Method

## 1.1 Principle

The spirit of VC-Q&A Method lies in the fact that civil engineers are mostly specialized in a particular field in civil engineering but in day-to-day work they have to face technical issues of all fields of civil engineering. For instance, a concrete engineer has to handle drainage works and foundation works in a construction project of pumping station.

VC-Q&A Method is devised to *improve the knowledge of engineers in different fields of civil engineering*. The knowledge in each field of civil engineering is presented in question-and-answer (Q&A) format to enable engineers to easily grasp the essence and key points of knowledge in particular civil engineering fields in the shortest time. The Q&A is not intended to cover ALL aspects of knowledge in a particular field of civil engineering. In fact, it only presents the key issues, knowledge and concerns and inspires engineers to study further on these topics. In short, it serves:

Purpose 1: Given limited available time, engineers could obtain the most and relevant knowledge in a particular field of civil engineering

Purpose 2: The Q&A presentation format induces engineers to look for further knowledge. The Q&A stimulates critical thinking of engineers, inspiring them to study further.

## 1.2 Features of VC-Q&A Method

The VC-Q&A Method involves the usage of this Manual to grasp the knowledge of different field(s) of civil engineering through Q&A style in the shortest time.

### *1.2.1 Different Civil Engineering Fields*

In VC-Q&A method, it includes **twelve** fields of civil engineering namely, bridge works, concrete works, drainage and sewage works, marine works, foundation, roadworks, slopes and earthwork, tunneling, site investigation, waterworks, steelworks and general issue.

Engineers choose the field(s) of engineering that they intend to improve their knowledge.

### *1.2.2 Objectives*

There is a list of objectives stated at the beginning of each Module. This is intended to give readers an rough idea the topics that are covered by the Module. An objective no. is assigned to each element and for those questions which fall into the ambit of this element, the objective no. shall appear at the end of the question. This helps readers identify the questions related to the objective in a easy manner.

### *1.2.3 Sub-topics under Civil Engineering Field*

In VC-Q&A method, the Q&A are well organized into sub-topics under a particular civil engineering field. For example, for bridge works it would be categorized into different parts as follow:

Part I: Bridge Design (Level One)
Part II: Construction Method (Level One)
Part III: Bearings and Expansion Joints (Level One)
Part I: Bridge Structure (Level Two)
Part II: Long Bridge (Level Two)
Part III: Prestressing Works (Level Two)

This allows engineer to select the sub-topics that they are mostly concerned and interested and get the required knowledge in the fastest manner.

### *1.2.4 Classification into Level*

In VC-Q&A method, the Q&A are classified into two different levels:

Level One – The Q&A contained in this level are core and essential knowledge that are highly recommended to engineers to read as a first step should they need to get familiar with this particular field of civil engineering.

Level Two - The Q&A contained in this level are more difficult and specialized.

Engineers are recommended to go through Level One as the first step, followed by Level Two if they have sufficient time.

### *1.2.5 Q&A Format*

All questions are designed and tailor-made based on the following rules:

(a) They are not common, simple and easy questions; instead most of the questions are frequently-asked-questions raised by engineers who mostly ail to find out the answers.
(b) They bring out the key issues and points of a particular subject
(c) The questions are those that a specialist in different fields may ask
(d) They allow engineers to understand why a particular subject is relevant in practice

There is flow from question to question and it is highly recommended to read the questions sequentially.

## 1.3 Learning Steps

Step 1. Select a Module (i.e. a particular field of civil engineering)

Step 2. Read through the Objectives to understand the topics that are covered by the Module.

Step 3. Select a Part in Level One

Step 4. Read all questions sequentially in the sub-topic

Step 5. Select a Part in Level Two

Step 6. Read all questions sequentially in the sub-topic

## 1.4 Merits of VC-Q&A Method

*(a) Effectiveness and Efficiency in Learning*
This method is a very effective and efficient in mastering different disciplines of civil engineering in shortest possible time. The question format is easy to read and digest and points out the key answers and

issues in a few sentences. It saves engineer's time to read numerous technical books and journals in order to get one answer.

*(b) Non-boring format*
The question format itself is more easy-to-read than engineering books full of monotonous paragraphs. It allows engineers to skip from one question to another which promotes reading of the whole Module/Sub-topic. Also, the manual incorporates difficult questions which are frequently raised by specialists who could not find out the answers. When reading through the questions, readers should be able to identify certain questions which has been raised by them previously without successfully looking up the answers.

*(c) Stimulation in Further Learning*
The questions itself are mostly raised by practicing engineers in that particular filed. As such, it enlightens readers and leads them to ask further questions.

# 2. Module One: Bridge Works

## Objectives

| Element | Description | Objective No. |
|---|---|---|
| **Bridge Design** | | |
| Bridge Form | Precast prestressed beams with in-situ concrete top slab | BF1 |
| | Multiple-cell box girder | BF2 |
| | Skew Bridge | BF3 |
| Span Arrangement | Continuous multiple-span | SA1 |
| | Simply supported multiple-span | SA2 |
| | Span ratio | SA3 |
| Bridge Deck Design | Sucker deck principle | BDD1 |
| | Grillage Analysis | BDD2 |
| | Null point Analysis | BDD3 |
| Bridge Abutment | Orientation of wing walls | BA1 |
| | Earth pressure on abutment | BA2 |
| General Design | Shear Lag | GD1 |
| | HA and HB Load | GD2 |
| **Construction Method** | | |
| Bridge Construction | Span-by-span construction | BC1 |
| | Incremental launching method | BC2 |
| | Balanced cantilever method | BC3 |
| Segmental Box Girder Bridges | Stitching | SBGB1 |
| | Match casting | SBGB2 |
| **Bearings and Expansion Joints** | | |
| Bearings Design | Orientation of bearings | B1 |
| | Preset | B2 |
| Types of Bearings | Pot bearing | TB1 |
| | Elastomeric bearing | TB2 |
| Expansion Joints | Joint continuity | EJ1 |
| **Bridge Structure** | | |
| Bridge Structure | Diaphragm | BS1 |
| | Transition slab | BS2 |

| Element | Description | Objective No. |
|---|---|---|
| | Split piers | BS3 |
| | Shear keys in abutment | BS4 |
| | Shock transmission unit | BS5 |
| | Abutment | BS6 |
| Truss | Vierendeel girder | T1 |
| | Warren Truss, Howe Truss and Pratt Truss | T2 |
| **Long Bridge** | | |
| Long Bridge Type | Cable-stayed bridges | LBT1 |
| | Suspension bridges | LBT2 |
| Aerodynamic Behaviour | Flutter | AB1 |
| | Vortex-induced vibrations | AB2 |
| **Prestressing Works** | | |
| Prestressing Type | External prestressing or internal prestressing | PT1 |
| | One-way prestressing or two-way prestressing | PT2 |
| Prestressing Component | Prestressing Reinforcement | PC1 |
| | Grout | PC2 |

# Level One (Core FAQs)

## ***Part I: Bridge Design***

**1. What are the main potential benefits in using the bridge form of precast prestressed beams supporting in-situ concrete top slab? (BF1)**

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.

**2. What are the potential advantages of continuous multiple-span deck over simply supported multiple-span deck? (SA1, SA2)**

Movement joints are normally added to bridge structures to accommodate movements due to dimensional changes arising from temperature variation, shrinkage, creep and effect of prestress. However, the provision of excessive movement joints should be avoided in design because movement joints always encounter problems giving rise to trouble in normal operation and this increases the cost of maintenance.

Some designers may prefer to add more movement joints to guard against possible occurrence of differential settlements. However, the effect of continuity is disabled by this excessive introduction of movement joints.

From structural point of view, the use of continuous deck enhances the reduction of bridge deck thickness. Moreover, deck continuity allows the potential increase in headroom in the mid-span of bridges by using sucker deck principle.

Some designers may prefer to employ the use of simply supported multiple-span deck to guard against possible occurrence of differential settlements. However, the effect of continuity is undermined by the introduction of movement joints. In essence, the structural reserve provided by a continuous bridge is destroyed by the multiple-span statically determinate structure resulting from the addition of joints.

Moreover, the reduction of joints in bridge structures represents substantial cost savings arising from the construction and maintenance costs of movement joints. The reduction of deck thickness helps to cut the cost for both the deck and foundation. In particular, the number of bearings in each piers is substantially reduced when compared with the case of simply supported multiple-span deck.

**3. 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? (SA1)**

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.

**4. What are the advantages of piers constructed monolithically with the bridge deck over usage of bearings?**

Basically, piers constructed monolithically with the bridge deck are advantageous in the following ways:

(i) Movement of the bridge deck is achieved by the bending deformation of long and slender piers. In this way, it saves the construction cost of bearings by using monolithic construction between bridge deck and piers. Moreover, it is not necessary to spend extra effort to design for drainage details and access for bearing replacement. On the other hand, in maintenance aspect substantial cost and time savings could be obtained by using monolithic construction instead of using bearings as bridge articulation.

(ii) Monolithic construction possesses the shortest effective Euler buckling length for piers because they are fixed supports at the interface between bridge deck and piers.

Note: Monolithic construction means that piers are connected to bridge decks without any joints and bearings.

**5. What is sucker deck principle for variable depth bridge decks? (BDD1)**

For a variable depth bridge deck, the depth of continuous multi-span bridge deck is increased in pier supports and this absorbs sagging moments in the mid-span with the consequent increase in hogging moments in pier supports. As a result, the mid-span depth can be significantly reduced due to the reduction in sagging moment. In essence, this sucker deck principle is applied in locations where headroom requirement is of great concern. Moreover, in terms of structural performance, sucker decks are effective in reducing dead loads than voided slab of equivalent uniform depth for span length between 20-40m. In terms of aesthetics point of view, the public tends to appreciate the structural form of arches and curved soffit rather than boring uniform deck alignment. Reference is made to Brian Pritchard (1992).

**6. Which type of multiple-cell box girder is better, cells connected by top flanges or cells connected both by top and bottom flanges? (BF2)**

When the depth of a box girder bridge exceeds 1/6 or 1/5 of the bridge width, 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 choice [56]. However, 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 multiple-cell box girders, there are generally two arrangements. The first one is that independent cells are connected by their top flanges only while the other one is that the cells are connected both at the top and bottom flanges. From the structural point of view, it is recommended to adopt the second arrangement. For the case of cells connected by top flanges only, their flanges are heavily stressed in the transverse direction owing to flexure which cannot be effectively distributed across the cross section.

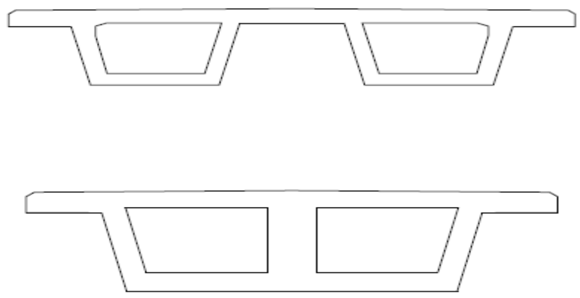

**Fig. Box girder with cells connected by top flanges and cells connected both by top and bottom flanges.**

### 7. What is the exact coverage of HA loading? (GD2)

Type HA loads first appeared in 1945 and the concept of HB load was introduced in BS 153 in 1954. Type HA loads is the normal loading for United Kingdom and covers vehicles up to 44 ton. HA loads are represented by a uniformly distributed load with a knife edge load. HA loads have covered the following situations:

(i) More than one vehicle occupying the width of a lane;
(ii) Overloading in normal vehicles;
(iii) Impact load induced when car wheel bounce when traveling crossing potholes.

### 8. Are knife edge loads the representation of wheel axles? (GD2)

In BS5400 the traffic loads for HA loading are given by the uniformly distributed loads along the loaded length and a knife edge load. In the code, it is not intended that knife edge loads simulate a wheel axle of vehicles [16]. Instead, it is just a tool to provide the same uniformly distributed loading to imitate the bending and shearing effects of actual traffic loads.

### 9. In grillage analysis of skew bridge, should a skew mesh be adopted or the transverse members are set orthogonal to the main members? (BF3)

For skew deck, the transverse members are set orthogonal to the main members to find out the correct moment and deflections. Skew decks develop twisting moments which is more severe for higher skew angles. The most economical way of designing reinforcing steel is to place the reinforcement along the direction of principal moment.

For skew angle less than $35^o$, it may not be practical to adopt this approach in the skew region. Instead, the transverse members are kept parallel to the support line so that a skewed mesh is adopted. The use of skew mesh suffers from the demerit that it would slightly overestimate the output of moment and deflections.

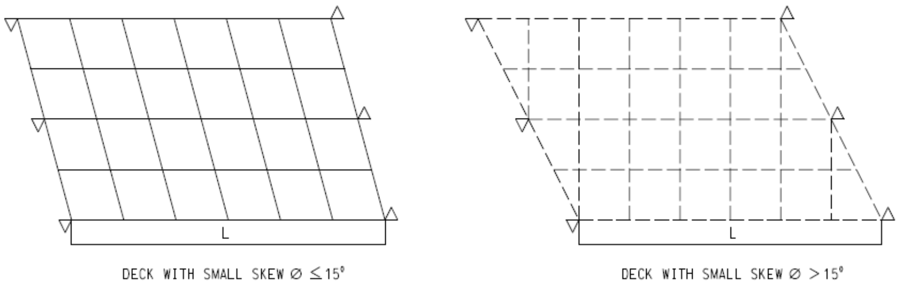


**Fig. Different reinforcing arrangement in skewed bridge**

## 10. What are the limitations of grillage analysis? (BDD2)

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.

**11. In the design of a simply supported skew bridge, which direction of reinforcement should be provided? (BF3)**

In the conventional design of steel reinforcement for a simply supported skew bridge, a set of reinforcement is usually placed parallel to free edge while the other set is designed parallel to the fixed edge. However, this kind of arrangement is not the most efficient way of placing the reinforcement. The reason is that in some parts of the bridge, the moment of resistance is provided by an obtuse angle formed by the reinforcement bars which is ineffective in resisting flexure. In fact, the most efficient way of the arrangement of reinforcement under most loading conditions is to place one set of bars perpendicular to the fixed edge while placing the other set parallel to the fixed end as recommended by L. A. Clark (1970). In this way, considerable savings would be obtained from the orthogonal arrangement of reinforcement.

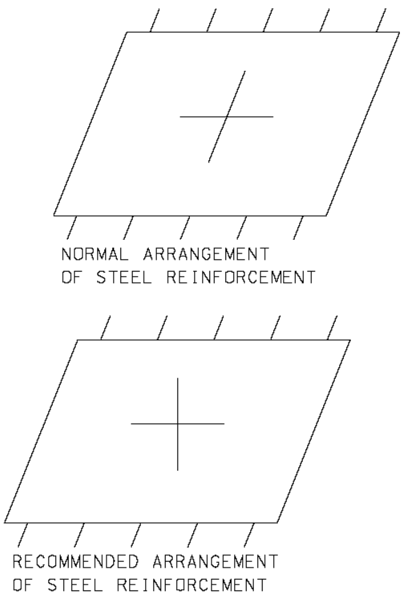


**Fig. The arrangement of reinforcement in skewed bridge.**

**12. Why is the span length ratio of end span/approach span to its neighboring inner spans usually about 0.75? (SA3)**

From aesthetic point of view, an odd number of spans with a decrease in length in the direction of abutment is desirable. Moreover, spans of equal length are found to be boring. However, the arrangement of irregular span lengths is not recommended because it gives a feeling of uneasiness.

From structural point of view, for a multi-span bridge with equal span length, the sagging moment at the mid-span of the end span/approach span is largest. In order to reduce this moment, the span length of end span/approach span is designed to be 0.75 of inner spans. However, this ratio should not be less than 0.40 because of the effect of uplifting at the end span/approach span support.

Note: End span refers to the last span in a continuous bridge while approach span refers top the first span of a bridge.

**13. What is the consideration in selecting the orientation of wing walls in the design of bridge abutments? (BA1)**

There are three common arrangements of wing walls in bridge abutments based on Dr. Edmund C Hambly (1979):

(i) Wing walls parallel to abutments
This is the simplest and shortest time to build but is not the most economical design. This design has the advantage that it has least disturbance to existing slope embankment.

(ii) Wing walls at an angle to abutments
This is the most economical design among the three options in terms of material cost.

(iii) Wing walls perpendicular to abutments
Though it is not the most economical design, the wing walls provide a continuous alignment with bridge decks which provide supports to parapets. However, they cause disturbances to adjacent structures and utility services during construction. Moreover, if the bridge is curved, the wing walls may hinder the road curvature.

One the other hand, when the wing walls are structurally connected to the abutment, then structural advantage can be taken by the stability of box structure.

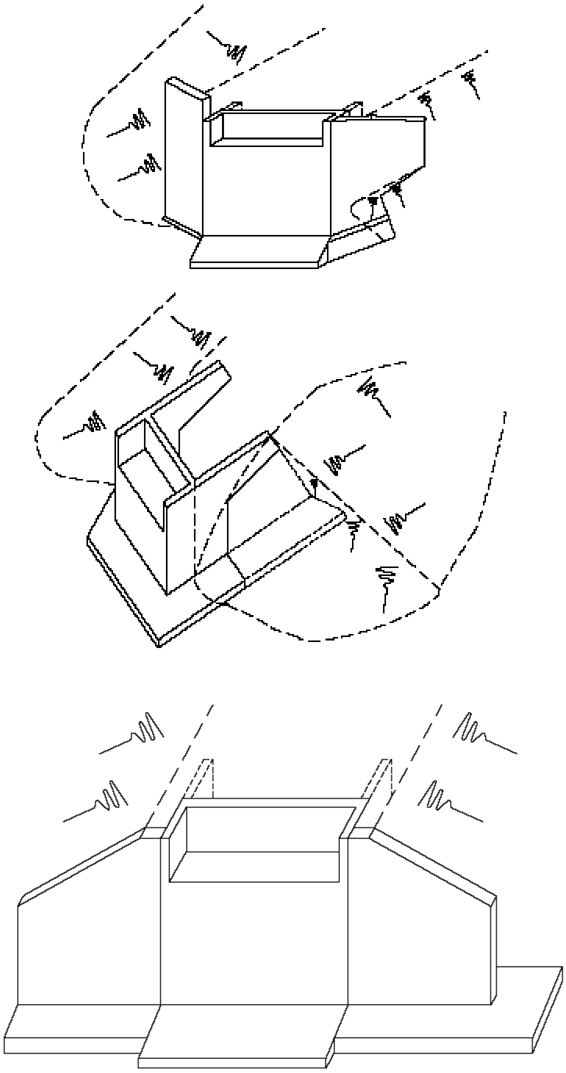

**Fig. Different orientation of wing walls.**

## 14. What is the significance of null point in bridge deck? (BDD3)

The null point is the position of zero movement in the bridge deck. When the bridge deck is pinned at a single pier, it provides the location of null point with no deck movement. However, when the bridge deck is pinned to more piers, the position of null point has to be calculated. The determination of null point is important because it serves to estimate the forces on the piers by deck length changes and to calculate the sliding movement of sliding and free bearings.

For symmetrical deck founded on two identical fixed piers, the null point should be midway between the two fixed piers. However, if one pier is taller than the other, the null point would be shifted to the shorter stiffer pier.

**15. What is the effect of shear lag in a typical box-girder bridge? (GD1)**

For multiple-cell box girders, there are generally two arrangements. The first one is that independent cells are connected by their top flanges only while the other one is that the cells are connected both at the top and bottom flanges. From the structural point of view, it is recommended to adopt the second arrangement. For the case of cells connected by top flanges only, their flanges are heavily stressed in the transverse direction owing to flexure which cannot be effectively distributed across the cross section.

In the structural analysis of bridges, shear lag have to be considered in design in some circumstances. Shear lag takes place when some parts of the cross section are not directly connected. For a box-girder bridge, not all parts of flanges are joined directly to webs so that the connected part becomes highly stressed while the unconnected flanges are not fully stressed. In particular, for wide flanges of box-girder bridges axial loads are transferred by shear from webs to flanges which result in the distortion in their planes. Consequently, the plane sections do not stay plane and the stress distribution in the flanges are not uniform. Moreover, there is a tendency for longitudinal in-plane displacements of bride deck away from the flange/web connection to lag behind those parts of the bridge in close vicinity to the flange/web connection.

The effect of shear lag causes the longitudinal stress at flange/web connection to be higher than the mean stress across the flange. Therefore, the effect of shear lag has to be catered for in the design of box-girder bridges, especially for those with wide flanges.

**16. How to estimate the earth pressure on abutment? (BA2)**

The magnitude of earth pressure coefficient in calculating the earth pressure on bridge abutment depends significantly on the degree of restraint provided by the abutment [30]. For example, active earth pressure is usually adopted for cantilever abutment because there is possible occurrence of small relieving movements. However, for abutment founded on piles, the at-rest earth pressure can be assumed in assessing the earth pressure as the abutment is considered to be rigidly supported by piles and is fully restrained against lateral movement.

**17. Should at-rest, active or passive soil pressure be used in the design of abutment? (BA2)**

At-rest soil pressure is developed during the construction of bridge abutment. Active soil pressure are developed when the abutment are pushed forward by backfilled soils at the back of abutment wall. A state of equilibrium shall be reached when the at-rest pressure is reduced to active earth pressure. Hence, at-rest pressure is considered when assessing the stability of abutment while active pressure is adopted when assessing the adequacy of structural elements of abutment.

Passive pressure is only considered in integral abutment which experiences passive pressure when the deck expands under thermal other effects.

Passive pressures are developed when the abutment wall pushes the soils at the front of abutment. Given that larger movements is required to mobilize passive pressure than active pressure and the abutment is designed not to slide under active pressure, it is normally assumed that passive pressure does not develop at the front of abutment. Moreover, there is a possibility that soils may be removed temporarily owing to utility diversion; it is normally assumed that stability contribution by soils in front of abutment is ignored.

**18. When would torsional stiffness of members be considered in analyzing a bridge?**

If a box-girder type bridge is purposely chosen because of its torsional strength, then the torsional stiffness and resistance should be considered in design. However, it is commonly accepted to assume that torsional stiffness of a beam to be negligible so that it saves the complexity to provide reinforcement to resist torsion. As such, this would result in higher bending moments induced in the beam.

If the torsional stiffness has been incorporated in computer model during the structural analysis, then it is necessary to check the torsional resistance of the beam.

**19. 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.

# Level One (Core FAQs)

## ***Part II: Construction Method***

**1. In the construction of a two-span bridge (span length = L) by using span-by-span construction, why is a length of about 1.25L bridge segment is constructed in the first phase of construction? (BC1)**

Basically, there are mainly three reasons for this arrangement:

(i) The permanent structure is a statically indeterminate structure. During construction by using span-by-span construction, if the first phase of construction consists of the first span length L only, then the sagging moment in the mid span of the partially completed bridge is larger than that of completed two-span permanent structure. To avoid such occurrence, 0.25L of bridge segment is extended further from the second pier which provides a counteracting moment, thereby reducing the mid-span moment of the partially completed bridge.

(ii) The position of 1.25 L countering from the first pier is the approximate location of point of contraflexure (assume that the two-span bridge is uniformly loaded) in which the bridge moment is about zero in the event of future loaded bridge. Therefore, the design of construction joint in this particular location has the least adverse effect on the structural performance of the bridge.

(iii) In case of a prestressed bridge, prestressing work has to be carried out after the construction of first segment of the bridge. If the prestressing work is conducted at the first pier which is heavily reinforced with reinforcement, it is undesirable when compared with the prestressing location at 1.25L from the first pier where there is relatively more space to accommodate prestressing works.

Note: Span-by-span construction means that a bridge is constructed from one bridge span to another until its completion.

**2. In span-by-span construction, which prestress layout is better (i) single-span coupled cable or (ii) two-span overlapped cable? (BC1)**

For single-span coupled cable, the length of cable is one span and they are coupled at the construction joint which is located at 0.25 of span. The use of single-span coupled cable in span-by-span construction suffers the following drawbacks:

(i) Stressing all tendons in one span is time consuming. Moreover, the construction team has to wait until the concrete has gained enough strength before all tendons in the span to be stressed.
(ii) Extra time is required for coupling of tendons.
(iii) The accommodation of coupler requires the lowering of designed tendon profile. Moreover, the coupler occupies large space in bridge web which is the region of high shear forces. To avoid generating a weak point in web, the web has to be locally thickened to maintain sufficient thickness of concrete.
(iv) Couplers have a higher risk of failure when compared with normal anchorages. The success of such prestress layout is highly dependent on the quality of coupler and workmanship because coupling of all prestressing tendons is carried out at the same point.
(v) The tendon length is only one span long which is economically undesirable.

For two-span overlapped cable, the cable is two-span long. At each construction phase in span-by-span construction, only 50% of tendons are stressed. In most cases, 50% tendons stressing would be sufficient to carry its self weight upon removal of falsework. As such, it allows the use of more economically longer cable with a reduction in construction time.

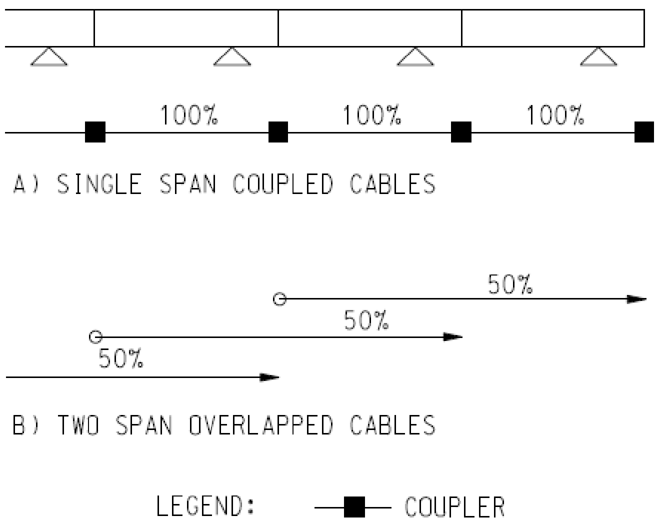


**Fig. Arrangement of single-span coupled cable and two-span overlapped cable**

**3. What is the difference between span-by-span construction and progressive placement? (BC1)**

Balanced cantilever construction simply cantilevers segments from a pier in a balanced manner on each side until the mid-span is reached and a closure is made with a previous half span cantilever from the preceding pier.

Progressive placement is similar to span-by-span construction as they both start from one end of structure to another. In progressive placement, precast segments are progressively placed in successive cantilevers on the same side of the same pier. This differs from the span-by-span method in which segments are cast at alternative sides of the same pier such that the hogging moment at the pier is counterbalanced. For progressive placement, temporary stays with temporary tower are placed on the pier to limit the cantilever stresses and deflections.

**4. Central prestressing is normally required during construction in incremental launching method. Why? (BC2)**

The erection condition plays an important role to the structural design of bridges when incremental launching method is adopted.

Each section of superstructure is manufactured directly against the preceding one and after concrete hardens, the whole structure is moved forward by the length of one section. When the superstructure is launched at prefabrication area behind one of the abutments, it is continually subjected to alternating bending moments. Each section of superstructure (about 15m to 25m long) is pushed from a region of positive moment and then to a region of negative moment and this loading cycled is repeated. As such, tensile stresses occur alternately at the bottom and top portion of superstructure section. For steel, it is of equal strength in both compression and tension and it has no difficulty in handling such alternating stress during launching process. However concrete could only resist small tensile stresses and therefore, central prestressing is carried out to reduce the tensile stress to acceptable levels.

Central prestressing means that the prestressing cables are arranged such that the resultant compressive stresses at all points in a given cross section are equal and it does not matter whether tensile stresses occur in upper or lower portion of superstructure during launching process.

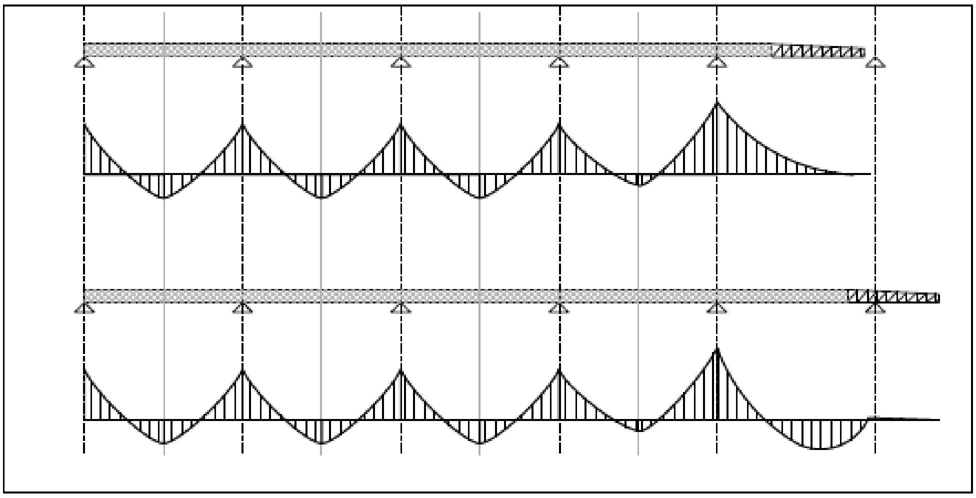

**Fig. Bending moment envelope in incremental launching**

**5. Which of the following methods to reduce cantilever moment is better in incremental launching, (i) temporary nose, (ii) mast or (iii) auxiliary piers? (BC2)**

The use of mast is an alternative to temporary nose. From practical point of view, the use of mast requires continual adjustment of forces in the guys when the superstructure is pushed forward. On the other hand, the implementation of temporary nose system does not require much attention during operation.

For large span lengths (>50m) it is advantageous to adopt auxiliary piers because this helps reduce the central prestress required. However, it may not be economical for auxiliary piers with height more than 40m.

**6. For incremental launching method, the span depth ratio of bridges is normally low. Why? (BC2)**

Bridges constructed by incremental launching method are usually low in span depth ratio and typical values are 14 to17. With low span depth ratio, the bridge segments are stiff in bending and torsion which is essential to cater for the launching process. Such low span depth ratio could tolerate the discrepancy in vertical alignment on supports over which they slide. Such differential settlements may occur owing to the shortening of piers when the superstructure slides over them and the differential deformation of different piers.

**7. In incremental launching method of bridge construction, what are the measures adopted to enhance sufficient resistance of the superstructure during the launching process? (BC2)**

(i) During the launching process the leading edge of the superstructure is subject to a large hogging moment. In this connection, steel launching nose typically about 0.6-0.65 times span length is provided at the leading edge to reduce the cantilever moment. Sometimes, instead of using launching nose a tower and stay system are designed which serves the same purpose.

(ii) The superstructure continually experiences alternative sagging and hogging moments during incremental launching. Normally, a central prestress is provided in which the compressive stress at all points of bridge cross section is equal. In this way, it caters for the possible occurrence of tensile stresses in upper and lower part of the cross section when subject to hogging and sagging moment respectively. Later when the whole superstructure is completely launched, continuity prestressing is performed in which the location and design of continuity tendons are based on the bending moments in final completed bridge condition and its provision is supplementary to the central prestress.

(iii) For very long span bridge, temporary piers are provided to limit the cantilever moment.

**8. Should special design be catered for in bridge piers upon jacking up of superstructure for installation of bearings in incremental launching method? (BC2)**

After the completion of launching process, the superstructure has to be lifted up to allow for installation of bearings. This is usually achieved by means of jacks to raise 5-10mm successively at each pier. In fact, it is anticipated that no special design is necessary for this operation because the effect of differential settlements at support should already be checked in bridge design. Level readings should be checked to ensure that it does not deviate from the designed figure.

**9. Can the use of temporary nose in incremental launching method reduce the cantilever moment of superstructure to the value of inner support moment? (BC2)**

When the superstructure is pushed forward, a temporary nose is usually adopted at the front end of the superstructure to reduce the cantilever moment for which the central prestress is designed. The length of temporary nose is about 60-65% of bridge span.

The bending moment of self-weight for internal spans (equal span) of long bridge is $-0.0833WL^2$ at piers and $+0.0417W L^2$ at mid-span (W = unit weight of deck and L = span length). However, without the use of temporary nose, the bending moment in the leading pier when the deck has to cantilever from one pier to another would be $-0.5WL^2$, which is 6 times higher than normal values at support.

Theoretically speaking, it is possible to reduce the cantilever moment to the value of inner support moment (i.e. $-0.0833WL^2$) with the use of a long nose. However, from economic point of view, it is would better to adopt temporary additional prestressing instead of longer nose. Hence, in actual site practice, the use of temporary nose would not reduce the cantilever moment of superstructure to the value of inner support moment but only to achieve $-0.105WL^2$.

**10. What are the main design considerations for temporary nose in incremental launching? (BC2)**

There are two main design considerations for temporary nose:

(i) Maximum sagging moment
The maximum sagging moment at the point of connecting the nose to superstructure occurs when the superstructure is launched far from the pier. It is estimated to occur at about 75% of the span length.

(ii) Maximum bearing pressure at bottom flange of temporary nose

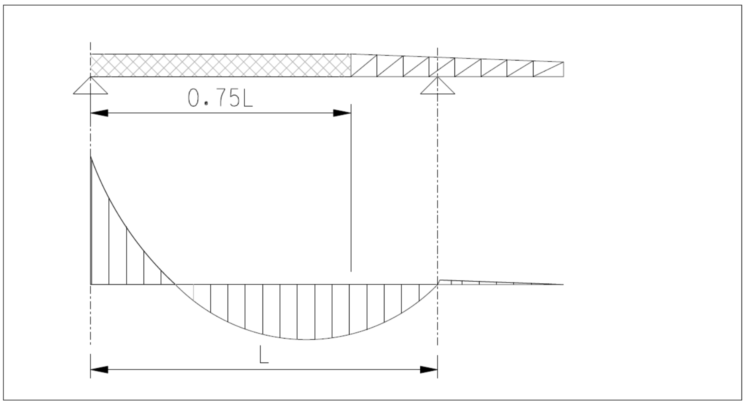


**Fig. Maximum sagging moment in temporary nose**

**11. Why is creep a major concern in balanced cantilever method? (BC3)**

In balanced cantilever method, the moment is balanced along the length of the piers. However, along the extended cantilevers only a part of negative bending moment is balanced by prestressing bending moment arising from normal force induced by prestressing. As such, it results in large deflections induced by concrete creep. These partly unbalanced permanent loads generate creep which produces unexpected and unevaluated hyperstatic effect. In fact, the compressive stresses are very high in lower slab while they are very low in upper slab of bridges.

**12. In bridge widening projects, the method of stitching is normally employed for connecting existing deck to the new deck. What are the problems associated with this method in terms of shrinkage of concrete? (SBGB1)**

In the method of stitching, it is a normal practice to construct the widening part of the bridge at first and let it stay undisturbed for several months. After that, concreting will then be carried out for the stitch between the existing deck and the new deck. In this way, the dead load of the widened part of bridge is supported by itself and loads arising from the newly constructed deck will not be transferred to the existing deck which is not designed to take up these extra loads.

One of the main concerns is the effect of stress induced by shrinkage of newly widened part of the bridge on the existing bridge. To address this problem, the widened part of the bridge is constructed a period of time (say 6-9 months) prior to stitching to the existing bridge so that shrinkage of the new bridge will take place within this period and the effect of shrinkage stress exerted on the new bridge is minimized.

Traffic vibration on the existing bridge causes adverse effect to the freshly placed stitches. To solve this problem, rapid hardening cement is used for the stitching concrete so as to shorten the time of setting of concrete. Moreover, the stitching work is designed to be carried out at nights of least traffic (Saturday night) and the existing bridge may even be closed for several hours (e.g. 6 hours) to let the stitching works to left undisturbed.

Sometimes, longitudinal joints are used in connecting new bridge segments to existing bridges. The main problem associated with this design is the safety concern of vehicles. The change of frictional coefficients of bridge deck and longitudinal joints when vehicles change traffic lanes is very dangerous to the vehicles. Moreover, maintenance of longitudinal joints in bridges is quite difficult.

Note: Stitching refers to formation of a segment of bridge deck between an existing bridge and a new bridge.

**13. In joints of precast concrete bridge segments, what are the functions of applying epoxy adhesive? (SBGB1)**

Epoxy adhesive is applied in these joints for the following purposes according to International Road Federation (1977):

(i) It seals up the joints completely between precast concrete segments to protect the prestressing tendons;
(ii) By filling voids and irregularities along the segment joints, it helps to reduce stress concentrations otherwise it will be developed; and
(iii) It helps in transferring of shear between the joints in case a large single shear key is used.

**14. In precast segmental box girder bridges, the bridge segments are usually formed by match casting. It is sometimes observed that a gap is formed between adjacent bridge segments. Why? (SBGB2)**

To enhance perfect fitting of bridge segments in precast segmental box girder bridges, segments are usually constructed by match casting so that it would not impair the serviceability and load bearing ability of the bridge. The end face of completed segment is adopted as formwork for the new segment. During the concrete hardening process, the hydration effect of new segment induces a temperature rise and develops a temperature gradient in the completed segment. Hence, the completed segment bows temporarily and the new segment sticks to this bowed shape when hardened. After match casting, the completed segment retains its original shape after cooling down while the new segment obtains the profile of bowed shape. Such bowing effect is even more significant for slender segments with large height to width ratio.

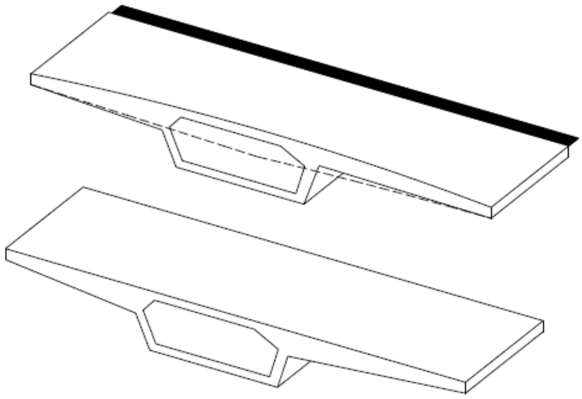

**Fig. Bowed shape**

## Level One (Core FAQs)

### ***Part III: Bearings and Expansion Joints***

**1. In a curved prestressed bridge, how should the guided bearings in piers of the curved region be oriented with respect to the fixed bearing in abutment? (B1)**

To determine the orientation of guided bearings, one should understand the movement of curved region of a prestressed bridge. Movement of prestress and creep are tangential to the curvature of the bridge (or along longitudinal axis) while the movement due to temperature and shrinkage effects are in a direction towards the fixed pier. If the direction of guided bearings is aligned towards the fixed bearing in the abutment, the difference in direction of pretress and creep movement and the guided direction towards fixed bearing would generate a locked-in force in the bridge system. The magnitude of the lock-in force is dependent on the stiffness of deck and supports. If the force is small, it can be designed as additional force acting on the support and deck. However, if the force is large, temporary freedom of movement at the guided bearings has to be provided during construction.

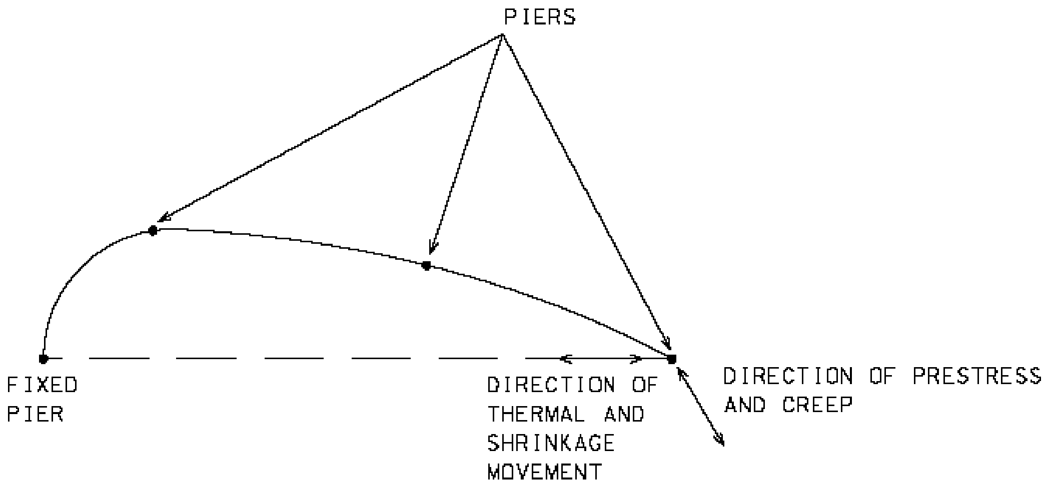


**Fig. The diagram showing how the guided bearings in piers of the curved region is oriented with respect to the fixed bearing in abutment.**

## 2. What is preset in bridge bearing? (B2)

"Preset" is a method to reduce the size of upper plates of sliding bearings in order to save cost. The normal length of an upper bearing plate should be composed of the following components: length of bearing + 2 x irreversible movement + 2 x reversible movement. Initially the bearing is placed at the mid-point of the upper bearing plate without considering the directional effect of irreversible movement. However, as irreversible movement normally takes place at one direction only, the bearing is displaced/presetted a distance of (irreversible movement/2) from the mid-point of bearing in which the length of upper plate length is equal to the length of bearing + irreversible movement + 2 x reversible movement. In this arrangement, the size of upper plate is minimized in which irreversible movement takes place in one direction only and there is no need to include the component of two irreversible movements in the upper plate.

Note: "Preset" refers to the displacement of a certain distance of sliding bearings with respect to upper bearing plates during installation of bearings.

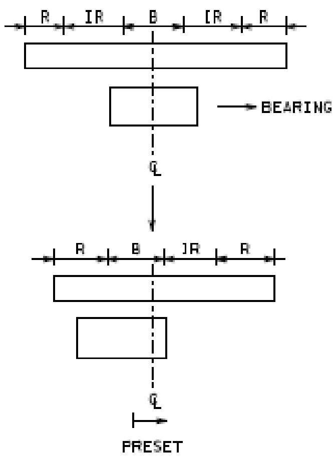


**Fig. Preset in sliding bearing.**

## 3. What is the purpose of leveling pad in bridge bearing?

Bridge bearings should be installed to lie horizontally on bridge piers and columns so that it would not induce eccentricity forces on substructure. However, the bridge superstructure requires different longitudinal and transverse level and gradient in order to keep in line with the geometry of the road. As such, it is natural to follow that the superstructure can hardly meet with substructure horizontally so that a leveling pad is introduced at the bottom of the superstructure to join with bridge bearing. Wedge-shaped leveling pad is commonly used for better concrete mobility at the bridge bearings.

**4. Under what situation should engineers use pot bearings instead of elastomeric bearings? (TB1)**

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 piers. Consequently, pot bearings are better alternatives than elastomeric bearings in such an scenario as suggested by David J. Lee.

**5. Why do most elastomers used in pot bearing are usually contained? (TB1)**

To specify space requirements, most pot bearings are designed for high contact pressures with small contact area with bridges. This also enhances lower friction values. Under the free state, most elastomers in pot bearings can hardly sustain this high pressure and hence they are most contained to prevent overstraining. When properly constrained, the elastomer behaves like semi-viscous fluid and can safely accommodate angular displacement.

**6. Polytetrafluoroethylene (PTFE) is commonly used in sliding bearings. Why? (TB1)**

The choice of sliding surface of bearings is of vital importance because the sliding surfaces generate frictional forces which are exerted on the bearings and substructure of the bridge. For instance, PTFE and lubricated bronze are commonly choices of sliding surfaces for bearings. PTFE is a flurocarbon polymer which possesses good chemical resistance and can function in a wide range of temperature. The most important characteristic of this material is its low coefficient of friction. PTFE has the lowest coefficients of static and dynamic friction of any solid with absence of stick-slip movement (David J. Lee). The coefficient of friction is found to decrease with an increase in compressive stress. However, PTFE do have some demerits like high thermal expansion and low compressive strength.

In designing the complementary contact plate with PTFE sliding surface, stainless steel plates are normally selected where the plates should be larger than PTFE surface to allow movement without exposing the PTFE. Moreover, it is recommended that the stainless steel surface be positioned on top of the PTFE surface to avoid contamination of dirt and rubbish. Lubricants are sometimes introduced to reduce the friction between the PTFE surface and the upper stainless steel plate. Hence, the PTFE may be designed with dimples to avoid the lubricant from squeezing out under repeated translation movements.

**7. What is the purpose of dimples in PTFE in bridge bearings? (TB1)**

PTFE is a flurocarbon polymer which possesses good chemical resistance and can function in a wide range of temperature. The most important characteristic of this material is its low coefficient of friction. PTFE has the lowest coefficients of static and dynamic friction of any solid with absence of stick-slip movement [43]. The coefficient of friction is found to decrease with an increase in compressive stress. However, PTFE do have some demerits like high thermal expansion and low compressive strength [43].

In designing the complementary contact plate with PTFE sliding surface, stainless steel plates are normally selected where the plates should be larger than PTFE surface to allow movement without exposing the PTFE. Moreover, it is recommended that the stainless steel surface be positioned on top of the PTFE surface to avoid contamination by possible accumulation of dirt and rubbish on the larger lower plates. Lubricants are sometimes introduced to reduce the friction between the PTFE surface and the upper stainless steel plate. Dimples are designed on PTFE surfaces to act as reservoirs for lubricant and these reservoirs are uniformly distributed over the surface of PTFE and normally they cover about 20%-30% of the surface area. Hence, the PTFE may be designed with dimples to avoid the lubricant from squeezing out under repeated translation movements.

**8. What is the purpose of dowel bar in elastomeric bearing? (TB2)**

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.

## 9. How to determine the size of elastomeric bearings? (TB2)

For elastomeric bearing, the vertical load is resisted by its compression while shear resistance of the bearing controls the horizontal movements. The design of elastomeric bearings is based on striking a balance between the provision of sufficient stiffness to resist high compressive force and the flexibility to allow for translation and rotation movement.

The cross sectional area is normally determined by the allowable pressure on the bearing support. Sometimes, the plan area of bearings is controlled by the maximum allowable compressive stress arising from the consideration of delamination of elastomer from steel plates. In addition, the size of elastomeric bearings is also influenced by considering the separation between the structure and the edge of bearing which may occur in rotation because tensile stresses deriving from separation may cause delamination. The thickness of bearings is designed based on the limitation of its horizontal stiffness and is controlled by movement requirements. The shear strain should be less than a certain limit to avoid the occurrence of rolling over and fatigue damage. The vertical stiffness of bearings is obtained by inserting sufficient number of steel plates.

## 10. In the design of elastomeric bearings, why are steel plates inserted inside the bearings? (TB2)

For elastomeric bearing to function as a soft spring, the bearing should be allowed for bulging laterally and the compression stiffness can 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.

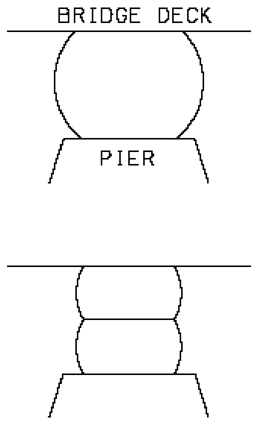


**Fig. Effect of steel plate in elastomeric bearing.**

**11. Why do some engineers prefer to use neoprene instead of natural rubber in elastomeric bearings? (TB2)**

Some engineers may choose to design elastomeric bearings to sit on the piers without a connection. The bearing is held in place by frictional resistance only. Paraffin used in natural rubber would bleed out and result in significant decrease in friction. As such, elastomeric bearings would slip away and walk out from their original locations. To solve this problem, neoprene, instead of natural rubber, is used as elastomer because paraffin is absent in neoprene bearings.

**12. What is the importance of shear stiffness in the design of elastomeric bearing? (TB2)**

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.

**13. For elastomeric bearings, which shape is better, rectangular or circular? (TB2)**

Circular bearings have the advantage for standardization because only one dimension can vary in plan. They are suitable for use in curved and large skewed bridge as they could accommodate movement and rotations in multiple directions.

Rectangular bearings are suitable for low skewed bridges. In particular, it is best suited in bridges with large rotations and movements.

**14. What is the advantage of sliding bearings over roller bearings?**

In roller bearing for a given movement the roller bearing exhibit a change in pressure centre from its original position by one-half of its movement based on David J. Lee. However, with sliding bearing a sliding plate is attached to the upper superstructure and the moving part of bearing element is built in the substructure. It follows that there is no change in pressure center after the movement.

**15. What is the importance of shear stiffness in the design of elastomeric bearing? (TB2)**

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.

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**16. Why are excessive movement joints undesirable in bridges? (EJ1)**

Movement joints are normally added to bridge structures to accommodate movements due to dimensional changes arising from temperature variation, shrinkage, creep and effect of prestress. However, the provision of excessive movement joints should be avoided in design because movement joints always encounter problems giving rise to trouble in normal operation and this increases the cost of maintenance.

Some designers may prefer to add more movement joints to guard against possible occurrence of differential settlements. However, the effect of continuity is disabled by this excessive introduction of movement joints. In essence, the structural reserve provided by a continuous bridge is destroyed by the multiple-span statically determinate structure resulting from the addition of excessive joints.

**17. How will inclined bridge deck affect joint continuity? (EJ1)**

Bearings are usually designed to sit in a horizontal plane so as to avoid the effect of additional horizontal force and uneven pressure distribution resulting from non-horizontal placing of bearings [43]. For an inclined bridge deck subject to a large longitudinal movement, a sudden jump is induced at the expansion joint and discontinuity of joint results. To solve this problem, an inclined bearing instead of a truly horizontal bearing is adopted if the piers can take up the induced horizontal forces.

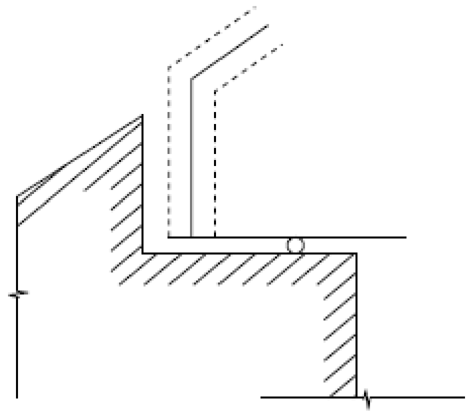

**Fig. The effect of inclined bridge deck on joint discontinuity.**

## 18. How does the position of bearing affect the continuity of joints?

Expansion joints in a bridge structures cater for movements in transverse, longitudinal, vertical and rotational forms. The layout and position of expansion joins and bearings have to be carefully designed to minimize the future maintenance problem.

The position of bearings affects the discontinuity of a joint [43]. If the location of a bearing is too far away from a bridge joint, discontinuity of the joint would be experienced when there is an excessive angular rotation at the joint. Hence, by keeping the bearings and movement joints close in position, the discontinuity in the vertical direction can be avoided.

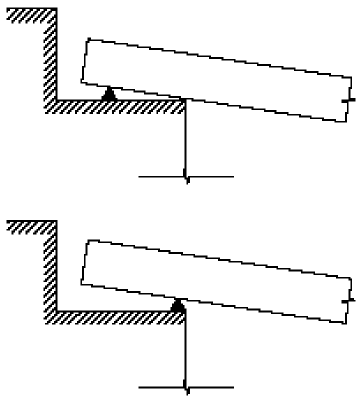

**Fig. The effect of position of bearing to the discontinuity of joint.**

# Level Two (Advanced FAQs)

## ***Part I: Bridge Structure***

### 1. What are the advantages of assigning the central pier and the abutment as fixed piers?

(i) For abutment pier to be assigned as fixed pier while the bridge is quite long, the longitudinal loads due to earthquake are quite large. As the earthquake loads are resisted by fixed piers, the size of fixed piers will be large and massive. In this connection, for better aesthetic appearance, the selection of abutment as fixed piers could accommodate the large size and massiveness of piers. Normally abutments are relatively short in height and for the same horizontal force, the bending moment induced is smaller.

(ii) For the central pier to be selected as the fixed pier, the bridge deck is allowed to move starting from the central pier to the end of the bridge. However, if the fixed pier is located at the abutment, the amount of movement to be incorporated in each bearing due to temperature variation, shrinkage, etc. is more than that when the fixed pier is located at central pier. Therefore, the size of movement joints can be reduced significantly.

### 2. What are the functions of diaphragms in bridges? (BS1)

Diaphragm is a member that resists lateral forces and transfers loads to support. Some of the diaphragms are post-tensioned and some contain normal reinforcement. It is needed for lateral stability during erection and for resisting and transferring earthquake loads. Based on past research, diaphragms are ineffective in controlling deflections and reducing member stresses. Moreover, it is commonly accepted that diaphragms aided in the overall distribution of live loads in bridges.

The main function of diaphragms is to provide stiffening effect to deck slab in case bridge webs are not situated directly on top of bearings. Therefore, diaphragms may not be necessary in case bridge bearings are placed directly under the webs because loads in bridge decks can be directly transferred to the bearings [56]. On the other hand, diaphragms also help to improve the load-sharing characteristics of bridges. In fact, diaphragms also contribute to the provision of torsional restraint to the bridge deck.

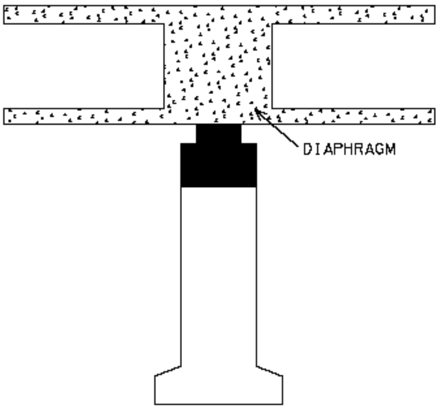


**Fig. Diaphragm.**

**3. Are diaphragms necessary in the design of concrete box girder bridges? (BS1)**

Diaphragms are adopted in concrete box girder bridges to transfer loads from bridge decks to bearings. Since the depth of diaphragms normally exceeds the width by two times, they are usually designed as deep beams. However, diaphragms may not be necessary in case bridge bearings are placed directly under the webs because loads in bridge decks can be directly transferred to the bearings based on Jorg Schlaich & Hartmut Scheef (1982). This arrangement suffers from the drawback that changing of bearings during future maintenance operation is more difficult.

In fact, diaphragms also contribute to the provision of torsional restraint to the bridge deck.

**4. How do engineer determine the number of cells for concrete box girder bridges?**

If 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 choice as suggested by Jorg Schlaich & Hartmut Scheef (1982). 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.

**5. What are the problems of using transition slabs in bridges? (BS2)**

In some designs, transition slabs are provided on the approach to bridges. For instance, soils in embankment supporting the roads may settle due to insufficient compaction and sharp depressions would be developed at the junction with the relatively rigid end of bridge decks [53]. This creates the problem of poor riding surfaces of carriageway and proper maintenance has to be carried out to rectify the situation. As a result, transition slabs are sometimes designed at these junctions to distribute the relative settlements between the approaching embankments and end of bridge decks so that the quality of riding surface between these junctions could be significantly improved and substantial savings could be obtained by requiring less maintenance.

**6. What is the purpose of overlays on concrete bridge deck?**

After years of servicing, some overlays may be applied on the top surface of bridges. Overlays on concrete bridge decks achieve the following purposes [8]:

(i) It aims to provide a smooth riding surface. Hence, it may be applied during the maintenance operation to hide the uneven and spalling deck surface and offers a smoother surface for road users.
(ii) The use of overlays can extend the life of the bridge deck.

**7. What are the purposes of waterproofing in bridge decks?**

Waterproofing materials like membranes are applied on top of bridge deck surface because:

(i) Vehicular traffic (e.g. tanker) may carry dangerous chemicals and the leakage of such chemicals in the absence of waterproofing materials may endanger the life of bridges. The chemicals easily penetrate and cause the deterioration of concrete bridge decks.
(ii) In some countries where very cold weather is frequently encountered, salt may be applied for defrosting purpose. In case waterproofing is not provided, the salt solution penetrates through the concrete cracks of the bridge and causes the corrosion of reinforcement.
(iii) In the event of cracks appearing on concrete deck, water penetrates the bridge deck and brings about steel corrosion.

**8. Why are split piers sometimes used when piers are built directly into the deck? (BS3)**

When the piers are built directly into deck without bearings, the monolithic construction creates a portal structure which modifies the bending moment envelope in the deck when compared with bridges with bearings. For instance, hogging moments are increased in supports with the decrease in sagging moments in mid-span of bridge deck. On the other hand, the shear stiffness of piers is a major concern because it tends to resist length changes of bridge deck which could not expand and contract readily.

In order to retain the bending stiffness of piers and to destroy the shear stiffness of pier simultaneously, the piers are split into two parts. The split pier act like the flange of an "I beam" which is effective in resist bending moment. The web of the "I beam", which is responsible for shear stiffness, is purposely removed so that the resulting split piers could deflect readily under length changes.

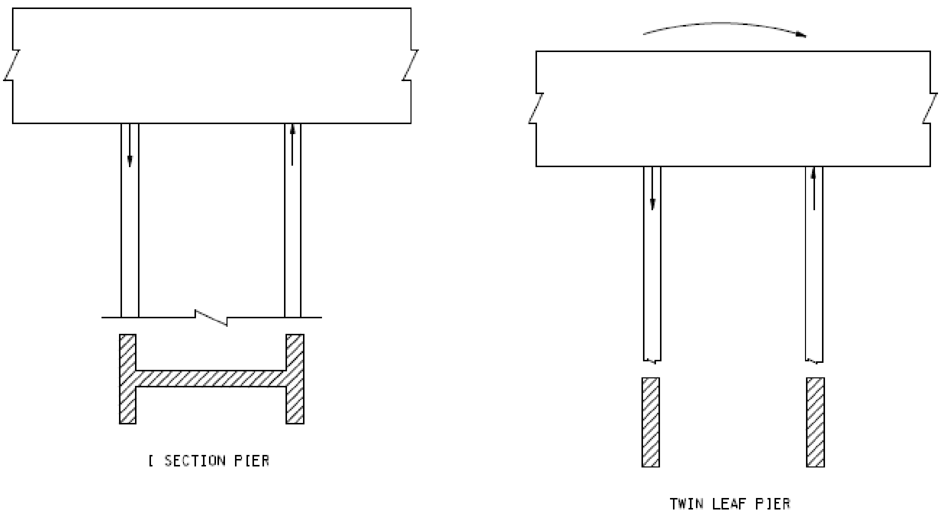


**Fig. Split pier**

**9. What is the significance of spacing of split piers? (BS3)**

Live loads on one span tend to cause uplift of outer column of the split piers (twin leaf piers). When the two split piers are designed too close, the uplift may be greater than the dead load reaction of the outer pier so that tension is induced in the outer pier. However, if the two split piers are designed to be adequately far apart, the uplift could hardly overcome the dead load leaving both piers in compression. As such, this allows for the use of pinned bearing. The optimum spacing of twin piers is about 1/25 to 1/13 of adjacent span.

**10. What is the purpose of providing a barrier around the bridge piers?**

Accidental collision of heavy vehicles such as tractor-trailer with bridge piers is not uncommon around the world. The consequence of such collision is catastrophic which may involve the collapse of bridges and loss of human lives. As such, suitable provisions are made to protect bridge piers against these accidental collisions. The most common way is to install a crashworthy barrier which should be designed to be capable of resisting the impact of heavy vehicles. Alternatively, in some countries such as the United States, they tend to revise the design of bridge barriers by requiring the bridge piers to be able to resist the collision of 1,800kN static force at 1.35m above ground.

**11. In bridge columns, why are stirrups be placed around the vertical reinforcement?**

In uniaxial compression test of concrete, upon reaching the ultimate load failure of concrete occurs where major cracks line up in the vertical direction and the concrete cube would be split up. The development of vertical cracks involves the expansion of concrete in lateral directions. In case the concrete is confined in lateral directions, it was observed that the formation of vertical cracks would be hindered as indicated in past experiments. As a result, the concrete strength is increased with also a rise in failure strain.

The above theory is often used in the design of bridge columns. Steel stirrups are installed at around the vertical main reinforcement. Other than the function of shear reinforcement, it helps to avoid the lateral deformation of interior concrete core so that the strength of concrete column is increased.

**12. What are the effects of bridge piers across a stream?**

The presence of bridge piers across a stream causes constricted flow in the openings because of the decrease of width of stream owing to the presence of the piers. Moreover, it creates the following problems from hydraulic point of view:

(i) Local scouring at the piers and bed erosion may take place. To avoid the damage to the foundation of piers, some protective layers of stone or concrete apron could be provided around the piers.
(ii) The head loss induced by the bridge piers causes the backwater effect so that the water level upstream is increased. Consequently, this may result in flooding in upstream areas.

**13. What is the mechanism of scouring at obstructions (e.g. bridge piers) in rivers?**

When the water flow in river is deflected by obstructions like bridge piers, scouring would occur arising from the formation of vortexes. The mechanism of formation of vortices is as follows: the flow hits the bridge piers and tends to move downwards. When the flow reaches the seabed, it would move in a direction opposite to its original flow direction before hitting the bridge piers. Hence, this movement of flow before the bridge piers results in the formation of a vortex. Owing to the formation of this vertical vortex, seabed material is continuously removed so that holes are formed at the seabed and this result in local scour at bridge piers. As the shape of vortices looks like horseshoes, it is sometimes called “horseshoe vortex”.

**14. What is the purpose of installation of shear keys in bridge abutment? (BS4)**

In small and medium sized bridges, shear keys are often designed in bridge abutments to provide transverse support to the bridge superstructure under lateral loads. They are not intended to carry vertical loads and they have important applications in resisting seismic loads.

Shear keys in bridge abutment are divided into two types, exterior or interior. Exterior shear keys have the demerit of the ease of inspection and repair. The shear keys are designed as sacrificial and it is assumed that once their capacity has been exceeded, the shear keys would not provide further support. As such, the bridge columns should be designed to provide transverse support once the shear keys fail to function.

**15. Which bridge parapet is better, steel parapet or aluminum parapet?**

Steel parapets normal requires painting or pre-treatment with hot-dip galvanizing as they are prone to corrosion and they are normally the cheapest choice for normal containment level of vehicles.

The initial material and setup cost of aluminum parapet is high. They are free of the problem of corrosion and the design of aluminum parapets does not require surface protection. However, owing to their high material price, care should be taken on the design to prevent stolen of parts of parapet. Moreover, aluminum parapet is lighter than steel and has weight savings over steel parapets.

**16. When should engineers consider using truss with K-bracing?**

In the arrangement of triangulated framework in truss structures, it is more economical to design longer members as ties while shorter ones as struts (e.g. Pratt truss). As such, the tension forces are taken up by longer steel members whose load carrying capacities are unrelated to their lengths. However, the compression forces are reacted by shorter members which possess higher buckling capabilities than longer steel members [34].

For heavy loads on a truss structure, the depth of the truss is intentionally made larger so as to increase the bending resistance and to reduce deflection. With the increase in length of the vertical struts, buckling may occur under vertical loads. Therefore, K-truss is designed in such as way that the vertical struts are supported by compression diagonals.

**17. What are the characteristics of Vierendeel girder? (T1)**

The Vierendeel girder design is sometimes adopted in the design of footbridges. In traditional truss design, triangular shape of truss is normally used because the shape cannot be changed without altering the length of its members. By applying loads only to the joints of trusses, the members of truss are only subjected to a uniform tensile or compressive stress across their cross sections because their lines of action pass through a common hinged joint.

The Vierendeel truss/girder is characterized by having only vertical members between the top and bottom chords and is a statically indeterminate structure. Hence, bending, shear and axial capacity of these members contribute to the resistance to external loads. The use of this girder enables the footbridge to span larger distances and present an attractive outlook. However, it suffers from the drawback that the distribution of stresses is more complicated than normal truss structures

[42].

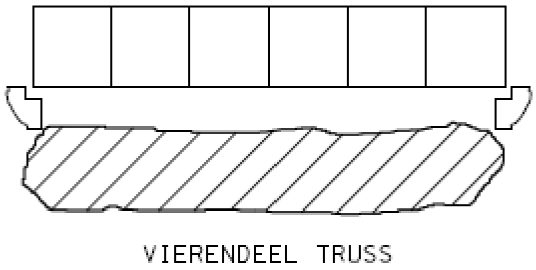


**Fig. Vierendeel Truss.**

**18. What are the differences among Warren Truss, Howe Truss and Pratt Truss? (T2)**

A truss is a simple structure whose members are subject to axial compression and tension only and but not bending moment. The most common truss types are Warren truss, Pratt truss and Howe truss. Warren truss contains a series of isoscele~~s~~ triangles or equilateral triangles. To increase the span length of the truss bridge, verticals are added for Warren Truss.

Pratt truss is characterized by having its diagonal members (except the end diagonals) slanted down towards the middle of the bridge span. Under such structural arrangement, when subject to external loads tension is induced in diagonal members while the vertical members tackle compressive forces. Hence, thinner and lighter steel or iron can be used as materials for diagonal members so that a more efficient structure can be enhanced.

The design of Howe truss is the opposite to that of Pratt truss in which the diagonal members are slanted in the direction opposite to that of Pratt truss (i.e. slanting away from the middle of bridge span) and as such compressive forces are generated in diagonal members. Hence, it is not economical to use steel members to handle compressive force.

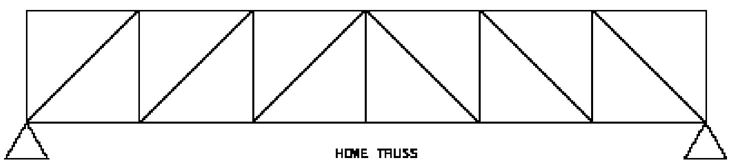


**Fig. A typical Howe Truss.**

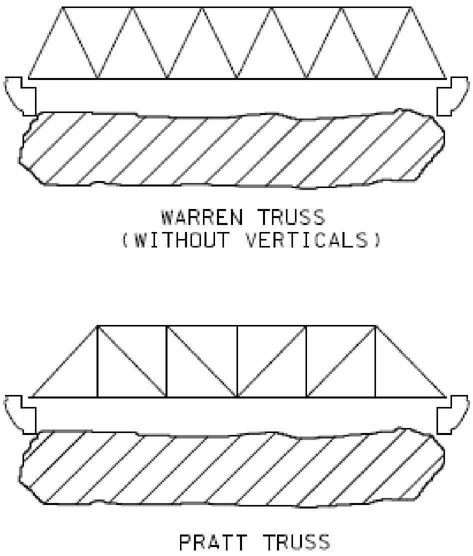


**Fig. Warren Truss and Pratt Truss**

### 19. What is the difference between dry joint and wet joint in precast segmental bridges?

Dry joints have been properly used in the past in which the bridge segments are formed by match casting. The prevalence in the past is due to it lower cost and time for construction. There is no gluing material to seal up the joint. As such, leakage through the joint into the box culvert occurs from time to time and this may affect the durability of external post-tensioning tendons. Moreover, owing to the effect of seismic, temperature and creep, the joints are found to open under these conditions. Spalling of top concrete slab at bridge joint was also reported.

Wet joint involves the use of epoxy glue at the mating precast segments. After the application of epoxy glue, a temporary precompression pressure of 0.3MPa is applied by stress bars at top, bottom and the sides of the mating precast segments. The epoxy sets under the applied pressure. The use of epoxy joints provides lubrication to help in the fit-up and alignment of the mating segments and minimizes the effect of hard point contact between segments.

**20. Why are precast concrete piers seldom used in seismic region?**

The use of precast concrete elements enhances faster construction when compared with cast-in-situ method. Moreover, it enhances high quality of piers because of stringent control at fabrication yards. The environmental impact is reduced especially for bridges constructed near waterways. In particular, for emergency repair of bridges owing to bridge collapse by earthquake and vehicular collision, fast construction of damaged bridge is of utmost importance to reduce the economic cost of bridge users.

The precast bridge piers are mostly used in non-seismic region but not in seismic region because of the potential difficulties in creating moment connections between precast members and this is essential for structures in seismic region.

**21. Why are coatings sometimes provided at the back faces of abutments? (BS6)**

There are different views on the necessity of the application of protective coatings (may be in the form of two coats of paint) to the back faces of bridge abutment [30]. The main purpose of this coating serves to provide waterproofing effect to the back faces of abutments. By reducing the seepage of water through the concrete, the amount of dirty materials accumulating on the surface of concrete would be significantly decreased. Engineers tend to consider this as an inexpensive method to provide extra protection to concrete. However, others may consider that such provision is a waste of money and is not worthwhile to spend additional money on this.

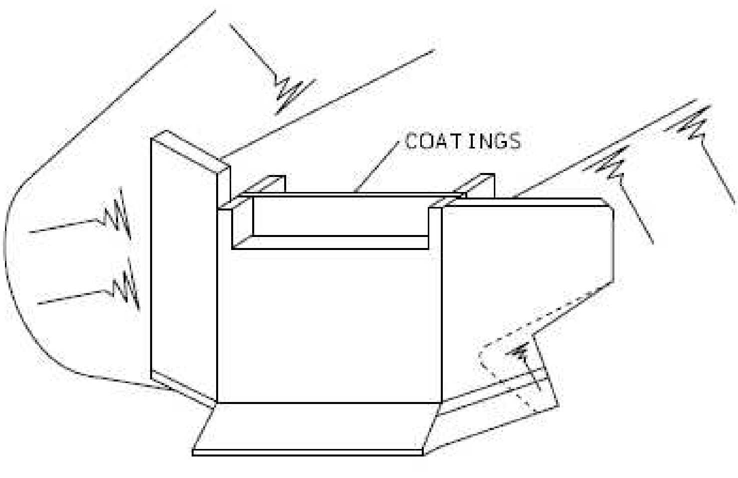


**Fig. Coatings at back faces of an abutment.**

## 22. What is shock transmission unit in bridges? (BS5)

Shock transmission unit is basically a device connecting separate structural units. It is characterized by its ability to transmit short-term impact forces between connecting structures while permitting long-term movements between the structures.

If two separate structures are linked together to resist dynamic loads, it is very difficult to connect them structurally with due allowance for long-term movements due to temperature variation and shrinkage effect [54]. Instead, large forces would be generated between the structures. However, with the use of shock transmission unit, it can cater for short-term transient loads while allowing long-term movements with negligible resistance. It benefits the bridge structures by acting as a temporary link between the structures to share and transfer the transient loads.

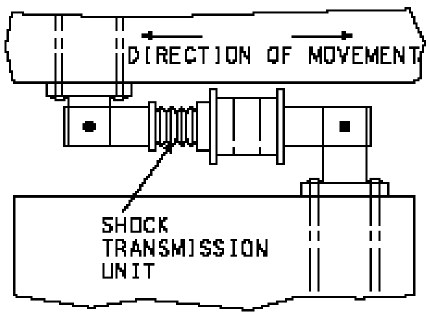


**Fig. Shock transmission unit.**

## 23. Should raking piles of a bridge abutment be placed under an embankment? (BS6)

For a bridge abutment to be supported on raking piles with different orientations, the movement between the ground and the pile group is difficult to predict. For instance, if some of the raking piles of the bridge abutment are extended beneath an embankment, then the settlement of embankment behind the abutment may cause the raking piles to experience severe bending moment and damage the piles as recommended by Dr. Edmund C Hambly (1979).

## 24. Sometimes the side of concrete bridges is observed to turn black in colour. What is the reason for this phenomenon?

In some cases, it may be due to the accumulation of dust and dirt. However, for the majority of such phenomenon, it is due to fungus or algae growth on concrete bridges. After rainfall, the bridge surface absorbs water and retains it for a certain period of time. Hence, this provides a good habitat for fungus or algae to grow. Moreover, atmospheric pollution and proximity of plants provide nutrients for their growth. Improvement in drainage details and application of painting and coating to bridges help to solve this problem. Reference is made to Sandberg Consulting Engineers Report 18

**25. Are there any problems associated with Integral Abutment Bridge? (BS6)**

Integral Abutment Bridges are bridges without expansion joints in bridge deck. The superstructure is cast integrally with their superstructure. The flexibility and stiffness of supports are designed to take up thermal and braking loads.

The design of Integral Abutment Bridges is simple as it may be considered as a continuous fame with a single horizontal member with two or more vertical members. The main advantage of this bridge form is jointless construction which saves the cost of installation and maintenance of expansion joints and bearings. It also enhances better vehicular riding quality. Moreover, uplift resistance at end span is increased because the integral abutment serves as counterweight. As such, a shorter end span could be achieved without the provision of holding down to expansion joints. The overall design efficiency is increased too as the longitudinal and transverse loads on superstructure are distributed over more supports.

However, there are potential problems regarding the settlement and heaving of backfill in bridge abutment. For instance, “granular flow” occurs in backfill materials and it is a form of on-going consolidation. Settlement of backfill continues with daily temperature cycles and it does not stabilize. Active failure of upper part of backfilling material also occurs with wall rotations. This leads to backfill densification and can aggravate settlement behind the abutment.

**26. What are the functions of sleepers in railway?**

The functions of sleepers [7] in railway works are as follows:

(i) The primary function of a sleeper is to grip the rail to gauge and to distribute the rail loads to ballast with acceptable induced pressure.
(ii) The side functions of a sleeper include the avoidance of both

longitudinal and lateral track movement.
(iii) It also helps to enhance correct line and level of the rails.

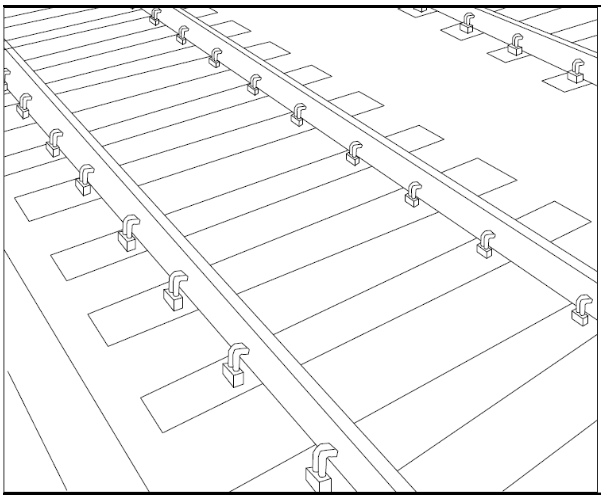

**Fig. Sleepers.**

# Level Two (Advanced FAQs)

## *Part II: Long Bridge*

### 1. What are the advantages of cable-stayed bridges over suspension bridges for span less than 1,000m? (LBT1)

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.

### 2. When is single plane or multiple plane used in cable-stayed bridges? (LBT1)

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.

### 3. What is the difference between gravity anchorage and tunnel anchorages in suspension bridges? (LBT2)

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.

**4. How does the shape of bridge deck affect the aerodynamic behaviour?**

Two types of bridge vibration that are of special concern are:

(i) Flutter, which is self-induced vibration characterized by occurrence of vertical and torsional motion at high wind speeds.
(ii) Vortex shedding, which is the vibration induced by turbulence alternating above and below the bridge deck at low wind speeds.

One of the important features affecting the aerodynamic behaviour of a bridge is the shape of bridge deck. The shape which provides maximum stability against wind effects is that of an airplane wing, on which the wind flows smoothly without creating turbulence and there is no separation of boundary layers. To improve the aerodynamic behaviour of a bridge, addition of wind fairings and baffle plates could be considered.

**5. How do vortex-induced vibrations affect the stability of long bridges? (AB2)**

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.

**6. How does flatter affect the stability of long bridges? (AB1)**

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.

**7. How does deck equipment (median dividers and parapets) affect the aerodynamic response of long-span bridges?**

Bridge parapets raise the overall level of bluffness of long-span bridges. When the solidity ratio of barriers increases, the effect of increasing the bluffness also becomes more significant. The principal effects of deck equipment such as median dividers and parapets is that it enhances an increase in drag forces and a reduction in average value of lift force.

**8. In wind tunnel test, why are similarity of Reynolds Number between real bridge and model is often neglected?**

Wind tunnel test is often conducted to check aerodynamic stability of long-span bridges. To properly conduct wind tunnel test, aerodynamic similarity conditions should be made equal between the proposed bridge and the model. Reynolds Number is one of these conditions and is defined as ratio of inertial force to viscous force of wind fluid. With equality of Froude Number, it is difficult to achieve equality in Reynolds Number.

For instance, for a model scale of 1/40 to 1/150, the ratio of Reynolds Number between the bridge and the model varies from 252 to 1837 with a difference of order from 100 to 1000. As such, similarity of Reynolds Number between real bridge and model is often neglected.

# Level Two (Advanced FAQs)

## ***Part III: Prestressing Works***

### 1. What are parasitic forces for prestressing?

In statically determinate structures, prestressing forces would cause the concrete structures to bend upwards. Hence, precambering is normally carried out to counteract such effect and make it more pleasant in appearance. However, for statically indeterminate structures the deformation of concrete members are restrained by the supports and consequently parasitic forces are developed by the prestressing force in addition to the bending moment generated by eccentricity of prestressing tendons [53]. The developed forces at the support modify the reactions of concrete members subjected to external loads and produces secondary moments (or parasitic moments) in the structure.

### 2. Why type of prestressing is better, external prestressing or internal prestressing? (PT1)

At several locations in the span (i.e. third or quarter points) the tendons are deviated to the correct tendon profile by concrete deviators in external prestressing. The advantages of external prestressing are listed below:

(i) Owing the absence of bond between the tendon and structure, external prestressing allows the removal and replacement of one or two tendon at one time so that the bridge could be retrofitted in the event of deterioration and their capacity could be increased easily. This is essential for bridges in urban areas where traffic disruption is undesirable.
(ii) It usually allows easy access to anchorages and provides the ease of inspection.
(iii) It allows the adjustment and control of tendon forces.
(iv) It permits the designer more freedom in selecting the shape of cross section of bridges.
(v) Webs could be made thinner so that there is a reduction of dead load.
(vi) It enhances a reduction of friction loss because the unintentional angular change like wobble is eliminated. Moreover, the use of polyethylene sheathing with external prestressing has lower friction coefficient than corrugated metal ducts in internal prestressing.

(vii) Improvement of concrete placing in bridge webs owing to the absence of ducts.

The major distinction between internal prestressing and external prestressing lies in the variation in cable eccentricity. The deflected shape of external tendons is not exactly the same as beams because the displacement of external tendons is controlled by deviators. This is a second order effect at working load and it is very important at ultimate load.

Based on past research, for small span with shallow cross section (i.e. less than 3m deep), the use of internal prestressing requires less steel reinforcement. However, for deeper bridge cross section, the employment of external prestressing results in smaller amount of steel reinforcement.

**3. Under what situation shall engineers use jacking at one end only and from both ends in prestressing work?**

During prestressing operation at one end, frictional losses will occur and the prestressing force decreases along the length of tendon until reaching the other end. These frictional losses include the friction induced due to a change of curvature of tendon duct and also the wobble effect due to deviation of duct alignment from the centerline. Therefore, the prestress force in the mid-span or at the other end will be greatly reduced in case the frictional loss is high. Consequently, prestressing, from both ends for a single span i.e. prestressing one-half of total tendons at one end and the remaining half at the other end is carried out to enable a even distribution and to provide symmetry of prestress force along the structure.

In fact, stressing at one end only has the potential advantage of lower cost when compared with stressing from both ends. For multiple spans (e.g. two spans) with unequal span length, jacking is usually carried out at the end of the longer span so as to provide a higher prestress force at the location of maximum positive moment. On the contrary, jacking from the end of the shorter span would be conducted if the negative moment at the intermediate support controls the prestress force. However, if the total span length is sufficiently long, jacking from both ends should be considered.

**4. Which one is better, one-way prestressing or two-way prestressing? (PT2)**

During prestressing operation at one end, frictional losses will occur and the prestressing force decreases along the length of tendon until reaching the other end. These frictional losses include the friction induced due to a change of curvature of tendon duct and also the wobble effect due to deviation of duct alignment from the centerline. Therefore, the prestress force in the mid-span or at the other end will be greatly reduced in case the frictional loss is high. Consequently, prestressing, from both ends for a single span i.e. prestressing one-half of total tendons at one end and the remaining half at the other end is carried out to enable a even distribution and to provide symmetry of prestress force along the structure.

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**5. What are the three major types of reinforcement used in prestressing? (PC1)**

(i) Spalling reinforcement
Spalling stresses are established behind the loaded area of anchor blocks and this causes breaking away of surface concrete. These stresses are induced by strain incompatibility with Poisson's effects or by the shape of stress trajectories.

(ii) Equilibrium reinforcement
Equilibrium reinforcement is required where there are several anchorages in which prestressing loads are applied sequentially.

(iii) Bursting Reinforcement
Tensile stresses are induced during prestressing operation and the maximum bursting stress occurs where the stress trajectories are concave towards the line of action of the load. Reinforcement is needed to resist these lateral tensile forces.

**6. Why is spalling reinforcement needed for prestressing works in anchor blocks? (PC1)**

Reinforcement of anchor blocks in prestressing works generally consists of bursting reinforcement, equilibrium reinforcement and spalling reinforcement. Bursting reinforcement is used where tensile stresses are induced during prestressing operation and the maximum bursting stress occurs where the stress trajectories are concave towards the line of action of the load. Reinforcement is needed to resist these lateral tensile forces. For equilibrium reinforcement, it is required where there are several anchorages in which prestressing loads are applied sequentially.

During prestressing, spalling stresses are generated in the region behind the loaded faces of anchor blocks [14]. At the zone between two anchorages, there is a volume of concrete surrounded by compressive stress trajectories. Forces are induced in the opposite direction to the applied forces and it forces the concrete out of the anchor block. On the other hand, the spalling stresses are set up owing to the strain compatibility relating to the effect of Poisson's ratio.

**7. What are the functions of grout inside tendon ducts? (PC2)**

Grout in prestressing works serves the following purposes:

(i) Protect the tendon against corrosion.
(ii) Improve the ultimate capacity of tendon.
(iii) Provide a bond between the structural member and the tendon.
(iv) In case of failure, the anchorage is not subject to all strain energy.

**8. In prestressing work, if more than one wire or strand is included in the same duct, why should all wires/strands be stressed at the same time? (PC1)**

If wires/strands are stressed individually inside the same duct, then those stressed strand/wires will bear against those unstressed ones and trap them. Therefore, the friction of the trapped wires is high and is undesirable.

**9. What is the optimum size of cable duct for prestressing? (PC1)**

The cross sectional area of duct is normally 2.5 times that of the area of prestressing steel. The size of ducts should be not designed to be too small because of the followings:

(i) Potential blockage by grout
(ii) Excessive development of friction
(iii) Difficulty in threading prestressing tendon

## 10. What is stress corrosion of prestressing steel? (PC1)

Stress corrosion is the crystalline cracking of metals under tensile stresses in the presence of corrosive agents [44]. The conditions for stress corrosion to occur are that the steel is subjected to tensile stresses arising from external loading or internally induced stress (e.g. prestressing). Moreover, the presence of corrosive agents is essential to trigger stress corrosion. One of the main features of stress corrosion is that the material fractures without any damage observed from the outside. Hence, stress corrosion occurs without any obvious warning signs.

# 3. Module Two: Concrete Works

## Objectives

| Element | Description | Objective No. |
|---|---|---|
| **Concrete Material** | | |
| Concrete Material | Definition of concrete | CM1 |
| | Cement | CM2 |
| | GGBS and PFA replacement | CM3 |
| | Aggregates | CM4 |
| Durability | Carbonation | D1 |
| | Shrinkage | D2 |
| **Concrete Structure** | | |
| Concrete Structure | Cover | CS1 |
| | Chamfer | CS2 |
| Precast Concrete | Lifting hoops | PC1 |
| | Lifting anchors | PC2 |
| **Construction of Concrete Structure** | | |
| Compaction | Vibrators | C1 |
| | Over-vibration | C2 |
| Pumping Concrete | Blockage | PC1 |
| Formation of Joints | Construction joint | FJ1 |
| Fresh Concrete | Effect of rain | FC1 |
| | Adding water/liquid nitrogen | FC2 |
| **Tests on Concrete** | | |
| Test | Compressive strength | T1 |
| | Schmidt hammer test | T2 |
| | Slump test | T3 |
| **Concrete Joint and Cracking Design** | | |
| Joint | Expansion/Contraction/Isolation joint | J1 |
| | Joint sealant | J2 |
| | Joint filler | J3 |
| | Waterstop | J4 |
| Cracks | Design | C1 |
| | Crack types | C2 |

| Element | Description | Objective No. |
|---|---|---|
| **Formwork and Curing** | | |
| Formwork | Release agent | F1 |
| | Formwork pressure | F2 |
| | Late removal | F3 |
| | Falsework | F4 |
| Curing | Curing compound | C1 |
| | Plastic sheet | C2 |
| **Steel Reinforcement** | | |
| Steel reinforcement | Spacing | SR1 |
| | Steel areas | SR1 |
| | Anchorage/Lap length | SR3 |
| | Rusting | SR4 |

# Level One (Core FAQs)

## ***Part I: Concrete Material***

### 1. Can grout replace concrete in normal structure?

The mixture of cement and water alone cannot replace concrete (Longman Scientific and Technical (1987)) because:

(i) Shrinkage of grout is several times that of concrete with the same mass.
(ii) The effect of creep of grout is far more than that of concrete.
(iii) Heat of hydration of cement with water is more than normal concrete and this leads to the problem of severe cracking.

### 2. What are the differences between epoxy grout, cement grout and cement mortar? (CM1)

Epoxy grout consists of epoxy resin, epoxy hardener and sand/aggregates. In fact, there are various types of resin used in construction industry like epoxy, polyester, polyurethane etc. Though epoxy grout appears to imply the presence of cement material by its name, it does not contain any cement at all. On the other hand, epoxy hardener serves to initiate the hardening process of epoxy grout. It is commonly used for repairing hairline cracks and cavities in concrete structures and can be adopted as primer or bonding agent.

Cement grout is formed by mixing cement powder with water in which the ratio of cement of water is more or less similar to that of concrete [63]. Owing to the relatively high water content, the mixing of cement with water produces a fluid suspension which can be poured under base plates or into holes. Setting and hardening are the important processes which affect the performance of cement grout. Moreover, the presence of excessive voids would also affect the strength, stiffness and permeability of grout. It is versatile in application of filling voids and gaps in structures.

Cement mortar is normally a mixture of cement, water and sand (typical proportion by weight is 1:0.4:3). It is intended that cement mortar is constructed by placing and packing rather than by pouring. They are used as bedding for concrete kerbs in roadwork. They are sometimes placed under base plates where a substantial proportion of load is designed to be transferred by the bedding to other members.

**3. What is the difference between no-fines concrete, lightweight concrete and lean concrete? (CM1)**

Pervious concrete is sometimes called "no fines" concrete. It is designed with high porosity and allows water to pass though. It is commonly used in concrete pavement so as to reduce surface runoff and allow the recharging of ground water. The high porosity is achieved by a network of interconnected void. "No fines" concrete has little or no fines and contains just enough cement paste to cover the surface of coarse aggregates while maintaining the interconnectivity of voids.

Lightweight concrete is characterized by low density (1,400kg/m$^3$ to 1,800 kg/m$^3$) and is made of lightweight coarse aggregates. In some cases, even the fine aggregates are also lightweight too. The primary use of lightweight concrete is to reduce the dead load of concrete structures.

Lean concrete, which is also known as cement bound material, has low cementitious material content. It has low concrete strength and is commonly used as roadbase material.

**4. What is the difference between foam concrete and cement grout? (CM1)**

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.

**5. What is the difference between "High strength concrete" and "High performance concrete"? (CM1)**

There is common confusion about the terms "High strength concrete" and "High performance concrete" and it appears that they refer to the same thing. In fact, "High performance concrete" refers to the concrete which has been specially designed to achieve a certain particular characteristics such as high abrasion assistance and compaction without segregation. It may turn out that the designed concrete would possess a high strength but this is obviously not the original intention.

"High strength concrete" is designed to have concrete strength of 70MPa and more.

**6. What is the difference between no-fines concrete, lightweight concrete and lean concrete? (CM1)**

Pervious concrete is sometimes called "no fines" concrete. It is designed with high porosity and allows water to pass though. It is commonly used in concrete pavement so as to reduce surface runoff and allow the recharging of ground water. The high porosity is achieved by a network of interconnected void. "No fines" concrete has little or no fines and contains just enough cement paste to cover the surface of coarse aggregates while maintaining the interconnectivity of voids.

Lightweight concrete is characterized by low density (1,400kg/m$^3$ to 1,800 kg/m$^3$) and is made of lightweight coarse aggregates. In some cases, even the fine aggregates are also lightweight too. The primary use of lightweight concrete is to reduce the dead load of concrete structures.

Lean concrete, which is also known as cement bound material, has low cementitious material content. It has low concrete strength and is commonly used as roadbase material.

**7. What is the purpose of adding gypsum in cement? (CM2)**

Gypsum is a mineral and is hydrated calcium sulfate in chemical form. Gypsum plays a very important role in controlling the rate of hardening of the cement. During the cement manufacturing process, upon the cooling of clinker, a small amount of gypsum is introduced during the final grinding process.

Gypsum is added to control the "setting of cement". If not added, the cement will set immediately after mixing of water leaving no time for concrete placing.

**8. Can rapid-hardening cement be used in water-retaining structures? (CM2)**

Normal Portland cement is usually adopted in water retaining structures. Where sulphates or chemical agents are anticipated in groundwater, sulphate-resisting cement may be used to guard against sulphate and chemical attack. However, it is normally not advisable to use rapid-hardening cement in water-retaining structures because it involves greater evolution of heat during hydration process, leading to increased shrinkage cracks which form the location of potential leakage in the structure. It is only applicable in cold weather condition where the rate of hydration is low.

**9. Is it desirable to use concrete of very high strength i.e. exceeding 60MPa? What are the potential problems associated with such high strength concrete?**

To increase the strength of concrete, say from 40MPa to 80MPa, it definitely helps in improving the structural performance of the structure by producing a denser, more durable and higher load capacity concrete. The size of concrete members can be significantly reduced resulting in substantial cost savings. However, an increase of concrete strength is also accompanied by the occurrence of thermal cracking. With an increase in concrete strength, the cement content is increased and this leads to higher thermal strains. Consequently, additional reinforcement has to be introduced to control these additional cracks caused by the increase in concrete strength. Moreover, the ductility of concrete decreases with an increase in concrete strength. Attention should be paid during the design of high strength concrete to increase the ductility of concrete. In addition, fire resistance of high strength concrete is found to be less than normal strength concrete as suggested by Odd E. Gjorv (1994).

Though the tensile strength of high strength concrete is higher than that of normal concrete, the rate of increase of tensile strength is not proportional to the increase of compressive strength. For normal concrete, tensile strength is about one-tenth of compressive strength. However, for high strength concrete, it may only drop to 5% of compressive strength.

Moreover, owing to a low aggregate content of high strength concrete, creep and shrinkage increases.

**10. What is the advantage of using GGBS as replacement of cement in concrete? (CM3)**

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.

However, it is identified that there are still some hindrances that prevent the prevalence of its usage in local market. Technically speaking, GGBS concrete suffers from lower rate of strength development which is highly sensitive to curing conditions. In this connection, certain site measures have to be introduced to the construction industry to ensure better quality of curing process in order to secure high quality of GGBS concrete. On the other hand, designers have to be cautious of the potential bleeding problem of GGBS concrete.

Another major hurdle of extensive use of GGBS concrete lies in the little source of supply of GGBS. As Hong Kong is not a major producer of steel, GGBS as a by-product of steel has to be imported overseas and this introduces higher material cost due to transportation and the supply of GGBS is unstable and unsteady.

**11. Which of the following cement replacement material is better, PFA or GGBS? (CM3)**

(i) Similarities

Both GGBS and PFA are by-products of industry and the use of them is environmentally friendly. Most importantly, with GBS and PFA adopted as partial replacement of cement, the demand for cement will be drastically reduced. As the manufacture of one tonne of cement generates about 1 tonne of carbon dioxide, the environment could be conserved by using less cement through partial replacement of PFA and GGBS.

On the other hand, the use of GGBS and PFA as partial replacement of cement enhances the long-term durability of concrete in terms of resistance to chloride attack, sulphate attack and alkali-silica reaction. It follows that the structure would remain to be serviceable for longer period, leading to substantial cost saving. Apart from the consideration of long-term durability, the use of PFA and GGBS results in the reduction of heat of hydration so that the problem of thermal cracking is greatly reduced. The enhanced control of thermal movement also contributes to better and long-term performance of concrete.

In terms of the development of strength, PFA and GGBS shared the common observation of lower initial strength development and higher final concrete strength. Hence, designers have to take into account the potential demerit of lower strength development and may make use of the merit of higher final concrete strength in design.

(ii) Differences Between GGBS and PFA

The use of GGBS as replacement of cement enhances smaller reliance on PFA. In particular, GGBS is considered to be more compatible with renewable energy source objectives.

The replacement level of GGBS can be as high as 70% of cement, which is about twice as much of PFA (typically replacement level is 40%). Hence, partial replacement of GGBS can enable higher reduction of cement content. As the manufacture of one tonne of cement generates about 1 tonne of carbon dioxide and it is considered more environmentally friendly to adopt GGBS owing to its potential higher level of cement replacement.

The performance of bleeding for GGBS and PFA varies. With PFA, bleeding is found to decrease owing to increased volume of fines. However, the amount of bleeding of GGBS is found to increase when compared with OPC concrete in the long term. On the other hand, drying shrinkage is higher for GGBS concrete while it is lower for PFA concrete.

In terms of cost consideration, the current market price of GGBS is similar to that of PFA. As the potential replacement of GGBS is much higher than PFA, substantial cost savings can be made by using GGBS.

**12. How does pulverized fly ash function as cement replacement? (CM3)**

Pulverized fly ash is a type of pozzolans. It is a siliceous or aluminous material which possesses no binding ability by itself. When it is in finely divided form, they can react with calcium hydroxide in the presence of moisture to form compounds with cementing properties.

During cement hydration with water, calcium hydroxide is formed which is non-cementitious in nature. However, when pulverized fly ash is added to calcium hydroxide, they react to produce calcium silicate hydrates which is highly cementitious. This results in improved concrete strength. This explains how pulverized fly ash can act as cement replacement.

**13. What are the functions of coating on concrete?**

In designing protective coating on concrete structures, stoppage of water ingress through the coating is normally required. Since chloride ions often diffuse into concrete in solution and cause deterioration of concrete structures, the prevention of water transmission into the coating certainly helps to protect the concrete structure. However, if water gets behind the coating from some means and becomes trapped, its effect may not be desirable. Firstly, vapour pressure would be developed behind the surface treatment and this leads to the loss of adhesion and the eventual peeling off of the coating. Moreover, the water creates a suitable environment for mould growth on concrete surface.

In fact, the surface treatment should be so selected that it is impermeable to liquid water but it is permeable to water vapour. This “breathing” function enhances the concrete to lose moisture through evaporation and reject the uptake of water during wet periods.

**14. What is the significance of Flakiness Index and Elongation Index?**

*Flakiness Index* is the percentage by weight of particles in it, whose least dimension (i.e. thickness) is less than three-fifths of its mean dimension.

*Elongation Index* is the percentage by weight of particles in it, whose largest dimension (i.e. length) is greater than one and four-fifths times its mean dimension.

Flaky and elongated particles may have adverse effects on concrete and bituminous mix. For instance, flaky and elongated particles tend to lower the workability of concrete mix which may impair the long-term durability. For bituminous mix, flaky particles are liable to break up and disintegrate during the pavement rolling process.

**15. Should cement and aggregates be measured by weight or by volume? (CM2)**

Measurement of constituents for concrete is normally carried out by weight because of the following reasons [55]:

(i) Air is trapped inside cement while water may be present in aggregates. As such measurement by volume requires the consideration of the bulking effect by air and water.
(ii) The accuracy of measurement of cement and aggregates by weight is higher when compared with measurement by volume when the weighing machine is properly calibrated and maintained. This reduces the potential of deviation in material quantity with higher accuracy in measurement for the design mix and leads to more economical design without the wastage of excess materials.

**16. What are the applications of no fines concrete? (CM1)**

In some occasions no fines concrete is used in houses because of its good thermal insulation properties. Basically no fines concrete consists of coarse aggregates and cement without any fine aggregates. It is essential that no fines concrete should be designed with a certain amount of voids to enhance thermal insulation. The size of these voids should be large enough to avoid the movement of moisture in the concrete section by capillary action. It is common for no fines concrete to be used as external walls in houses because rains falling on the surface of external walls can only penetrate a short horizontal distance and then falls to the bottom of the walls. The use of no fines concrete guarantees good thermal insulation of the house.

**17. What is the difference between dry-mix method and wet-mix method in shotcrete?**

Shotcrete is not a special product of concrete and it is not a special method of placing concrete.

In dry-mix method the dry cementitious mixture is blown through a hose to the nozzle, where the water is injected immediately. The dry-mix method appears to be better for low volume placements. The nozzleman should pay great attention in adding the necessary amount of water during shooting operation.

Wet-mix method involves the pumping of ready mixed concrete to the nozzle. Compressed air is introduced at the nozzle to impel the mixture onto the receiving surface. The wet-mix method is more favorable for large volume placements. Rebound is less than in the dry-mix method. The nozzleman does not need to be concerned with the control on water addition.

**18. What is the difference between carbonation and carbon dioxide attack? (D1)**

For carbon dioxide attack, carbon dioxide dissolves in water to form a weak acid called carbonic acid. It would dissolve the cement matrix. However, the amount of carbon dioxide from the atmosphere is usually not sufficient to cause harm to concrete structures until additional source of carbon dioxide is available (e.g. decaying vegetable matter).

Carbonation is the process of converting alkaline hydroxides in concrete to carbonates by reaction with carbon dioxide. The significance of carbonation lies in the reduction of pH of pore water in concrete structure from 12-13 to 8-9 so that it drops the protection to steel reinforcement. The process takes place at concrete surface and spreads inwards. The passive nature play an important role in steel corrosion as it prevents corrosion even in the presence of water and oxygen. This passive nature is derived from a stable and thin layer of iron oxide formed at the surface of steel reinforcement. However, if the pH of concrete is dropped, this passive oxide layer becomes unstable and corrosion may start once water and oxygen supply is available.

Stress would be generated owing to the occurrence of shrinkage. For instance, concrete at surface tends to dry more rapidly than the interior, leading to differential shrinkage. This results in the formation of internal balancing forces in which there is tension on concrete surface with compression in the interior concrete. About 90% of ultimate concrete strain occurs in the first year.

When the water is allowed to absorb water, only part of shrinkage is reversible. Water loss from gel pore with C-S-H and capillary pres can be readily replenished. However, water loss from the space between C-S-H layers cannot be easily replaced with water. Once water is lost from the space between C-S-H layers at the first place, the bond between the layers becomes stronger and the layers would get closer together, thus making it difficult for water to re-enter into the reduced space.

**19. What is the mechanism of plastic settlement in fresh concrete?**

Within a few hours after the placing of fresh concrete, plastic concrete may experience cracking owing to the occurrence of plastic shrinkage and plastic settlement. The cause of plastic settlement is related to bleeding of fresh concrete. Bleeding refers to the migration of water to the top of concrete and the movement of solid particles to the bottom of fresh concrete. The expulsion of water during bleeding results in the reduction of the volume of fresh concrete. This induces a downward movement of wet concrete. If such movement is hindered by the presence of obstacles like steel reinforcement, cracks will be formed.

**20. What are the differences in the behavior and properties of recycled-aggregate concrete when compared to normal-aggregate concrete? (CM4)**

Higher porosity of recycled aggregate compared to natural aggregate leads to a higher absorption. Moreover, recycled aggregate has lower specific gravity than natural aggregate and will make concrete with higher drying shrinkage and creep. Such differences become more significant when there is an increasing amount of recycled fine aggregates. Recycled aggregates also contain more chloride than normal aggregates which may affect the durability of concrete. Moreover, excessive recycled fine aggregates can also generate a harsh concrete mix with low workability.

**21. Why are recycled aggregates and recycled concrete aggregates not suitable for "High strength concrete"? (CM4)**

Not all aggregates are suitable for producing "High strength concrete" because the strength of aggregates may control the ultimate strength of concrete once they are crushed apart before the failure of cement paste.

Recycled aggregates and recycled concrete aggregates are normally not recommended in producing "High strength concrete". Owing to their intrinsic effect of reduced compressive strength, it requires increased cement content to counterbalance this effect in normal concrete situation. However, in the case of "High strength concrete", the very high cement level has already been adopted which offers little scope for further increase in cement content.

**22. Is shrinkage in concrete a totally reversible process? (D2)**

In drying shrinkage, the excessive water which has not taken part in hydration process would migrate from interior of concrete core to the concrete surface. As a result of evaporation of the water moisture, the volume of concrete shrinks. The reduction in volume owing to moisture loss is termed shrinkage. In fact, aggregates in concrete would not cause shrinkage and helps to resist the deformation.

# Level One (Core FAQs)

## ***Part II: Concrete Structure***

### 1. What is the main purpose of chamfers in concrete structures? (CS2)

Some would suggest aesthetics as the answer of the question. Others may consider safety on the contrary because chamfered edge is less liable to damage to the concrete structure and any objects hitting on it. However, the main reason of the provision of chamfer is to make formworks easier to pull out after concreting operation. It is not uncommon that concrete is getting adhered to formwork and tore away during removal of formwork. In fact, the formation of a sharp nice corner is practically difficult and the concrete at corners is easily chipped and broken into pieces during the removal of formwork. Hence, the provision of chamber could protect the corner from chipping when striking formwork.

### 2. If a contractor proposes to increase concrete cover beyond contractual specification (i.e. 40mm to 70mm), shall engineers accept the proposal? (CS1)

In contractual aspect, based on the requirement of General Specification of Civil Engineering Works (1992 Edition), the tolerance of concrete cover is between +5mm and –5mm and engineers should not accept sub-standard work because they do not possess the authority to change the acceptance criteria. In case engineers consider contractor's proposal acceptable in technical point of view, consent has to be sought from the employer regarding the changes in acceptance criteria.

From technical point of view, the effect on cracking due to an increase in concrete cover should be considered. In general, there are three main parameters which govern crack width, namely tensile strain at the point considered, the distance of longitudinal bar to the concerned point and the depth of tension zone.

For the second factor, i.e. proximity of longitudinal bars to point of section, the closer a bar is to this point, the smaller is the crack width. Therefore, closely spaced bars with smaller cover will give narrower cracks than widely spaced bars with larger cover. Therefore, with an increase of concrete cover, the crack width will increase which is undesirable.

**3. How does concrete cover enhance fire resistance? (CS1)**

In the event of exposing the concrete structures to a fire, a temperature gradient is established across the cross section of concrete structures. For shallow covers, the steel reinforcement inside the structures rises in temperature. Generally speaking, steel loses about half of its strength when temperature rises to about $550^{o}C$. Gradually, the steel loses strength and this leads to considerable deflections and even structural failure in the worst scenario. Hence, adequate cover should be provided for reinforced concrete structure as a means to delay the rise in temperature in steel reinforcement.

**4. In designing concrete structures, normally maximum aggregate sizes are adopted with ranges from 10mm to 20mm. Does an increase of maximum aggregate size benefit the structures?**

To answer this question, let's consider an example of a cube. The surface area to volume ratio of a cube is 6/b where b is the length of the cube. This implies that the surface area to volume ratio decreases with an increase in volume. Therefore, when the size of maximum aggregate is increased, the surface area to be wetted by water per unit volume is reduced. Consequently, the water requirement of the concrete mixes is reduced accordingly so that the water/cement ratio can be lowered, resulting in a rise in concrete strength.

However, an increase of aggregate size is also accompanied by the effect of reduced contact areas and discontinuities created by these larger sized particles. In general, for maximum aggregate sizes below 40mm, the effect of lower water requirement can offset the disadvantages brought about by discontinuities as suggested by Longman Scientific and Technical (1987).

**5. What the preferable size of cover blocks? (CS1)**

The purpose of cover blocks are:

i. Maintain the required cover.
ii. Prevent steel bars from getting exposed to the atmosphere so that steel corrosion may result.
iii. Place and fix reinforcement based on design drawings.

As cover blocks after concreting shall form part of concrete structure, it is preferably that the cover blocks shall possess similar strength to the concrete structure. Moreover, the size of cover block should be minimized so that the chances of water penetration to the periphery would be reduced.

**6. Do engineers need to cater for corrosion protection of lifting anchors in precast concrete units?**

The corrosion of lifting anchors in precast concrete units has to be prevented because the corroded lifting units cause an increase in steel volume leading to the spalling of nearby surface concrete. Consequently, steel reinforcement of the precast concrete units may be exposed and this in turns results in the corrosion of steel reinforcement and the reduction in the load carrying capacity of the precast units. To combat the potential corrosion problem, the lifting anchors could be covered with a layer of mortar to hide them from the possible external corrosion agents. Alternatively, galvanized or stainless steel lifting anchors can be considered in aggressive environment.

**7. What are the potential problems of excessive concrete covers? (CS1)**

In reinforced concrete structures cover is normally provided to protect steel reinforcement from corrosion and to provide fire resistance. However, the use of cover more than required is undesirable in the following ways [25]:

(i) The size of crack is controlled by the distance of longitudinal bars to the point of section under consideration. The closer a bar is to this point, the smaller is the crack width. Therefore, closely spaced bars with smaller cover will give narrower cracks than widely spaced bars with larger cover. Consequently, with an increase in concrete cover the crack width will increase.
(ii) The weight of the concrete structure is increased by an increase in concrete cover. This effect is a critical factor in the design of floating ships and platforms where self-weight is an important design criterion.
(iii) For the same depth of concrete section, the increase of concrete cover results in the reduction of the lever arm of internal resisting force.

**8. Is mild steel or high yield steel suitable as lifting hoops in precast concrete? (PC1)**

The strength of high yield steel is undoubtedly higher than mild steel and hence high yield steel is commonly used as main steel reinforcement in concrete structures. However, mild yield steel is commonly used in links or stirrups because they can be subjected to bending of a lower radius of curvature.

For lifting hoops in precast concrete, it is essential that the hoops can be bent easily and hence mild steel is commonly adopted for lifting hoops because high yield bars may undergo tension cracking when it is bent through a small radius.

**9. Where is the desirable location of lifting anchors in precast concrete units? (PC2)**

It is desirable that the position of anchors be located symmetrical to the centre of gravity of the precast concrete units. Otherwise, some anchors would be subject to higher tensile forces when compared the other anchors depending on their distance from the centre of gravity of the precast concrete units. As such, special checks have to be made to verify if the anchor bolts are capable of resisting the increased tensile forces.

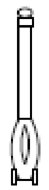


**Fig. A typical concrete anchor.**

**10. What is the purpose of installation of resilient bearings in buildings?**

When railway tunnels are built close to buildings, ground-borne vibration is transmitted to the building by means of compression and shear waves [36]. When structural members (e.g. wall) of a building have natural frequencies similar to the frequency at the source, the response of the structures would be magnified. This effect is even more significant when the building is designed with small number of movement joints. Consequently, the vibration can be felt inside the building and noise associated with such vibration is produced. To avoid this, vibration isolation can be implemented, sometimes by providing resilient bearings at column heads in buildings.

## 11. What is the function of cladding in concrete buildings?

Cladding refers to the external layer of the building which provides the aesthetic effect. Apart from the external appearance of the building, the main use of cladding is to protect the building structure from weather in one of the following ways:

(i) Cladding is made of impermeable materials so that rain water could only access the building through joints. However, properly designed joints with sealant could completely keep out the rain infiltration.
(ii) Cladding is made up of porous material (bricks) which absorbs rain during rainfall and subsequently dries out. Water could hardly penetrate into the building given that the cladding is of sufficient thickness with low permeability.

## 12. What is the purpose of applying spatterdash before rendering and plastering?

Spatterdash is a mixture of one part of cement to one and a half parts of coarse sand with enough water. The mixture is thrown forcibly onto the wall so that the impact removes the water film at the interface between spatterdash and the substrate leading to the improvement in adhesion. The spatterdash should cover the substrate surface completely and form a rough texture. Spatterdash serves as *an effective mechanical key* to prevent rendering and plastering material from sliding or sagging. The roughness of spatterdash improves adhesion by providing a positive "key" for plaster to grip. The improper application of spatterdash affects the subsequent bonding of rendering with substrate.

## 13. What is the purpose of adding polyethylene film in interior slabs sitting on grade for building structures?

Membrane materials, such as polyethylene film**,** are commonly used to reduce vapour transmission from soils to concrete slab. They are often termed "vapour retarder" and are placed on the underside of concrete slabs sitting directly on soils for building structures. Protection from moisture is essential because floors inside buildings are normally covered with carpet and tiles and penetration of water vapour through concrete slabs could result in the failure of adhesives in tiling, discoloration of flooring products and fungal growth.

**14. What is the mechanism of formation of pedestrian level winds around buildings?**

When a building blocks the wind blowing across it, part of the wind will escape over the top of the building. Some will pass around the edges of the building while a majority of the wind will get down to the ground. The channeling effect of wind for an escaping path, together with the high wind speeds associated with higher elevations, generates high wind speeds in the region at the base of the building. At the base level of the building, there are three locations of strong pedestrian level winds:

(i) Arcade passages – wind flow is generated by the pressure difference between the front and the back of the building.
(ii) At the front of the building – high wind is produced by standing vortex.
(iii) At the corners of the building – high wind is induced by corner flow.

# Level One (Core FAQs)

## ***Part III: Construction of Concrete Structure***

### 1. Should large-diameter or small-diameter vibrators be used in compacting concrete? (C1)

There is a general rule regarding the size of vibrators in compacting concrete. The diameter of the vibrator should be a quarter of the wall thickness of the concrete being cast. In general, large-diameter internal vibrators have higher amplitude with lower frequency while small-diameter internal vibrators have lower amplitude with higher frequency. In particular, small-diameter internal vibrators with high frequency are normally used in compacting high slump concrete.

### 2. What is the problem of over vibration of fresh concrete? (C2)

For proper compaction of concrete by immersion vibrators, the vibrating part of the vibrators should be completely inserted into the concrete. The action of compaction is enhanced by providing a sufficient head of concrete above the vibrating part of the vibrators. This serves to push down and subject the fresh concrete to confinement within the zone of vibrating action.

Over-vibration should normally be avoided during the compaction of concrete. If the concrete mix is designed with low workability, over-vibration simply consumes extra power of the vibration, resulting in the wastage of energy. For most of concrete mixes, over-vibration creates the problem of segregation in which the denser aggregates settle to the bottom while the lighter cement paste tends to move upwards [40]. If the concrete structure is cast by successive lifts of concrete pour, the upper weaker layer (or laitance) caused by segregation forms the potential plane of weakness leading to possible failure of the concrete structure during operation. If concrete is placed in a single lift for road works, the resistance to abrasion is poor for the laitance surface of the carriageway. This becomes a critical problem to concrete carriageway where its surface is constantly subject to tearing and traction forces exerted by vehicular traffic.

### 3. What are the problems if reinforcement is in contact with internal vibrators? (C1)

During concreting, if internal vibrators are placed accidentally in contact with some of the reinforcement bars, some undesirable effects may result. The most obvious one is that the reinforcement bars may become damaged or displaced if loosely tied.

Air bubbles tend to move towards the source of vibration. For poker vibrators touching the reinforcement bars, air pockets may be trapped in the vicinity of the reinforcement because the vibration generated by internal vibrators attracts these air bubbles. Consequently, the bond between the reinforcement and surrounding concrete would be impaired.

To produce good surface finish close to densely-packed reinforcement cage, workers may insert the poker vibrators in the gap between the reinforcement cage and formwork because the reinforcement cage tend to damp down the vibration effect when the vibrators are placed at a distance from the formwork. However, concentrated vibration within the cover region causes the migration of finer cement mortar to this region and results in changes in concrete colour. If the concrete cover is small, the chance of getting the poker vibrators jammed within the gap is high and the formwork is likely to be damaged by the vibrators.

**4. What are the reasons for blockage in pumping concrete? (PC1)**

Concrete pumping is commonly adopted in highly elevated locations for which access for concrete trucks is difficult. Construction works can be speeded up by using concrete pumping because a larger volume of pours can be achieved within a specified duration when compared with normal concrete placing methods.

Blockage may occur during pumping operation for the following two common reasons [18]:

(i) For saturated concrete mixes, the pump pressures may force water out of the concrete resulting in bleeding. The flow resistance is then increased and may contribute to the blockage of pipelines.

(ii) If the cement content (or other components of concrete mixes that increase the frictional forces) is high, a higher frictional resistance to pumping may develop and the concrete may not be pumpable.

**5. How does rain affect the freshly placed concrete? (FC1)**

Rain may affect the water cement ratio at top portion of freshly placed concrete provided that the concrete is not properly protected from rain. To substantially change the water-cement ratio of the concrete at the surface of the slab, *external energy must be supplied* to the system such as troweling passes with excess water on the concrete surface. The energy supplied by the finishing operations pushes the excess water into the slab surface creating a high water cement ratio in the near surface of the concrete so that its strength and durability is reduced. Sometimes, the damage to the concrete surface is apparent since the texture of the surface is easily damaged after the initial curing period. When the surface strength is affected, the long-term durability of the concrete may be reduced. *However, the concrete strength and durability below the surface would not be affected.*

**6. Why is liquid nitrogen added to fresh concrete sometimes instead of ice block/chilled water? (FC2)**

Traditionally, chilled water and ice have been employed to reduce the temperature of concrete mix in hot weather condition. Chilled water has a limitation in its cooling potential. For instance, even if all mixing water has been converted into chilled water, the temperature reduction achieved in concrete mix is only about 2.7 $^{o}$C. The complete/partial replacement of mixing water with ice may be a better alternative in terms of cooling potential because ice possesses power from heat of fusion. However, it presents practical difficulty in ensuring homogeneous distribution of ice within concrete mix and the complete melting of ice. Unmelted ice block may be hidden in concrete mix and if it melts before concrete setting it creates high water cement ratio locally. In case unmelted ice block melts after concrete hardening, large voids would be formed which impairs the concrete strength and durability.

Liquid nitrogen is supercooled and has a very high cooling potential. Fresh concrete can be cooled inside a read-mix truck by injection of liquid nitrogen. Liquid nitrogen is kept at a temperature of -196$^{o}$C in storage tank. Once liquid nitrogen is added to fresh concrete mix, nitrogen in liquid form changes to gaseous state under normal atmospheric pressure in a very short time (e.g. milliseconds).

**7. What are the potential problems of using liquid nitrogen to cool concrete? (FC2)**

Based on the results of past research, there is minimal impact on the properties and performance of concrete by liquid nitrogen (LN). The addition of liquid nitrogen appears to decrease the slump value of fresh concrete. However, the slump loss is not caused by liquid nitrogen but by previous hot concrete mix, i.e. the slump of LN-cooled concrete is the same of original hot concrete. Liquid nitrogen can also be observed to extend the setting time of concrete.

The safety of workers is one of the major concerns when using liquid nitrogen. The extremely low temperature of liquid nitrogen is dangerous to workers as prolonged contact of liquid nitrogen with skin cause severe burns and frostbite. Most concrete mixing drums may not be capable to endure the thermal shock brought about by liquid nitrogen.

**8. Should on-site addition of water to fresh concrete be allowed? (FC2)**

The addition of water to fresh concrete in truck mixer upon arrival at the location of concrete is allowed only if some design mixing water is held back during initial mixing stage. Addition of water in excess of design mixing water would definitely cause an increase of water cement ratio leading to a reduction of concrete strength.

The purpose of adding water to fresh concrete is to increase the workability to facilitate placement of concrete. The use of water-reducing agent or superplasticizer could help resolve the problem but extra attention has to be paid *on the segregation issue*. In case there is some buffer in the amount of water in fresh concrete so that addition of water would not result in exceeding the designed water cement ratio, water could be added based on the following rule of thumb: *5 liters of water per $m^3$ of concrete gives about 25mm increase in slump*.

**9. What are the reasons in setting maximum and minimum time for concreting successive lift in water-retaining structures?**

Maximum time for concreting successive lifts is required for concrete structures to reduce the potential differential strains. However, minimum time is needed to allow for possible shrinkage and thermal contraction to take place in the event of alternative bay construction before concreting to alternate bays. Otherwise, cracking may result which is undesirable in water retaining structure.

**10. Why shouldn't internal vibrators be forced down into fresh concrete? (C1)**

Internal vibrators operate by generating impulses which liquefy the fresh paste so that the internal friction between aggregates is reduced. As a result of vibration effect, the mix become unstable and trapped air would rise to the top while aggregates would settle to the bottom.

Internal vibrators should be allowed to fall in fresh concrete under its own weight and should not be forced to drag down into concrete. The reason behind this is that the use of force to push down internal vibrators would leave a mortar channel in fresh concrete and this result in the formation of weak concrete along this channel.

**11. What is the function of rebate in a typical construction joint? (FJ1)**

Construction joints are created on sites to facilitate the construction process. However, if improperly constructed, the completed construction joints will leave an uneven scar on the concrete surface and affect significantly its appearance. To avoid this, a rebate is formed during the first pour of one side of construction joint. After the other pour is concreted, it will hide the uneven joint inside the rebate.

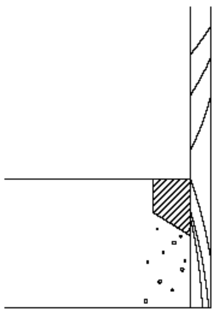

**Fig. A rebate.**

**12. Where are the desirable locations of construction joints? (FJ1)**

Construction joints are normally required in construction works because there is limited supply of fresh concrete in concrete batching plants in a single day and the size of concrete pour may be too large to be concreted in one go.

The number of construction joints in concrete structures should be minimized. If construction joints are necessary to facilitate construction, it is normally aligned perpendicular to the direction of the member. For beams and slabs, construction joints are preferably located at about one-third of the span length. The choice of this location is based on the consideration of low bending moment anticipated with relatively low shear force [10]. However, location of one-third span is not applicable to simply supported beams and slabs because this location is expected to have considerable shear forces and bending moment when subjected to design loads. Sometimes, engineers may tend to select the end supports as locations for construction joints just to simplify construction.

**13. What are the potential problems for retardation of fresh concrete?**

Retardation of fresh concrete has several advantages as follows:

(i) A rapid hydration process results in loss in concrete strength because the concrete will have a poorer structure with a higher gel/space ratio compared with the concrete with a lower hydration rate.
(ii) During the hydration process, ~~a~~ substantial heat of hydration will be generated. If the hydration process is carried out too swiftly, it will cause a rapid rise in temperature and result~~s~~ in considerable early thermal movement in concrete.
(iii) In hot weather concreting, the loss of workability is substantial. In order to ensure sufficient compaction of fresh concrete, it is necessary to extend the time for fresh concrete to remain plastic.

**14. What are the major problems in using pumping for concreting works? (PC1)**

In pumping operation, the force exerted by pumps must overcome the friction between concrete and the pumping pipes, the weight of concrete and the pressure head when placing concrete above the pumps. In fact, as only water is pumpable, it is the water in the concrete that transfers the pressure.

The main problems associated with pumping are the effect of segregation and bleeding. To rectify these adverse effects, the proportion of cement is increased to enhance the cohesion in order to reduce segregation and bleeding. On the other hand, a proper selection of aggregate grading helps to improve the pumpability of concrete.

# Level One (Core FAQs)

## ***Part IV: Tests on Concrete***

### 1. Some concrete specification requires the testing of compressive strength for both 7 days and 28 days. Why? (T1)

7-day compressive strength test results are usually not used for acceptance purpose *but for information only*. Instead, 28-day compressive strength test results are commonly adopted for acceptance purpose.

7-day compressive strength test results are often used to monitor the gain of early strength and they are estimated to be about 64% to 70% of the 28-day strength. As such, it serves as a warning signal to both concrete producers and contractors should the 7-day compressive strength test results are far less than 75% of the 28-day strength. Nowadays, most concrete placement schedule are very tight and it is of paramount importance for contractors to get to know as soon as possible the occurrence of low 7-day compressive strength test results. As such, the contractor could implement suitable measures promptly to get better quality control procedures at construction site and to monitor closely on sampling, molding, and testing of the test cubes so as to avoid the recurrence of the production of low-strength concrete in the coming concrete batches.

### 2. Is Schmidt hammer test a standard test for testing concrete strength? (T2)

The Schmidt hammer test involves hitting the in-situ concrete with a spring-driven pin at a defined energy, and then the rebound is measured. The rebound depends on the surface hardness of the concrete and is measured by test equipment. By referring to the some conversion tables, the rebound result of the test can be used to determine the compressive strength of the concrete. Although past investigations showed that there is a general relationship between compressive strength of concrete and the rebound number, there is a wide range of disagreement among various research workers regarding the accuracy of estimation of strength from Schmidt hammer. In fact, there is about a variation of 15-20% in concrete strength measured by the method.

Schmidt hammer is in not a standard test for acceptance testing of concrete strength. It is only a test used for estimating the strength of concrete in structure and it can hardly be considered as a substitute for compressive strength test.

**3. Is compressive strength test required for standard mixes of concrete? (T1)**

In some countries like Britain, specification for concrete does not normally require cube tests for standard mixes of concrete. The quality control of standard mixes in Britain is achieved by checking if the appropriate mix proportions are adopted during the mixing of concrete. However, in Hong Kong the requirement of testing for compressive strength is still required for standard mixes in the specification because it is impractical to inspect and check all constituent materials (e.g. cement, aggregates etc.) for concrete for compliance. As there is high variability in mixing materials owing to variance in the origin of production of constituent materials in Hong Kong, there is a risk that the end-product concrete does not comply with the design requirements even though the mix proportions of standard mixes are followed closely by engineers.

**4. In concrete compression test, normally 150mmx150mmx150mm concrete cube samples is used for testing. Why isn't 100mmx100mmx100mm concrete cube samples used in the test instead of 150mmx150mmx150mm concrete cube samples? (T1)**

Basically, the force supplied by a concrete compression machine is a definite value. For normal concrete strength application, say below 50MPa, the stress produced by a 150mmx150mmx150mm cube is sufficient for the machine to crush the concrete sample. However, if the designed concrete strength is 100MPa, under the same force (about 2,000kN) supplied by the machine, the stress under a 150mmx150mmx150mm cube is not sufficient to crush the concrete cube. Therefore, 100mmx100mmx100mm concrete cubes are used instead to increase the applied stress to crush the concrete cubes.

For normal concrete strength, the cube size of 150mmx150mmx150mm is already sufficient for the crushing strength of the machine.

**5. If concrete compression test fails, should Schmidt hammer test be adopted as an alternative test to prove the concrete strength? (T1)**

The Scmidt hammer test is based on the elastic rebound of hammer which presses on concrete surface and it measures the surface hardness of concrete. Since the test is very sensitive to the presence of aggregates and voids at the concrete surface, it is necessary to take more than 10 readings over the area of test. However, it should be noted that Schmidt hammer test measures surface hardness only but not the strength of concrete. Therefore, it may not be considered a good substitute for concrete compression test.

**6. In carrying out compression test for concrete, should test cubes or test cylinders be adopted? (T1)**

Basically, the results of compression test carried out by using cubes are higher than that by cylinders. In compression test, the failure mode is in the form of tensile splitting induced by uniaxial compression. However, since the concrete samples tend to expand laterally under compression, the friction developed at the concrete-machine interface generates forces which apparently increase the compressive strength of concrete. However, when the ratio of height to width of sample increases, the effect of shear on compressive strength becomes smaller. This explains why the results of compression test by cylinders are lower than that of cubes. Reference is made to Longman Scientific and Technical (1987).

**7. Is slump test a good test for measuring workability? (T3)**

Though slump test is originally designed as a measure of workability, it turns out to be an indicator of excessive water content in concrete only.

Slump test is not considered as a measure of workability because:

(i) There is no connection between the test results of slump test and workability;
(ii) The test results exhibit large random variations which is greater than that due to observed differences in workability;
(iii) Concrete of different workability may have the same slump.

**8. In “High strength concrete” in buildings, 56 or 91-day compression test results are sometimes adopted instead of 28-day compression test results. Why? (T1)**

In normal concrete structures, 28-day test results are often adopted. However, in the construction of high-rise buildings using “High strength concrete”, compressive strengths based on 56 or 91-day compression test results are commonly used instead. Since the process of construction of high-rise buildings involves the construction of lower levels firstly in which they are not loaded for a period of a year and more. Substantial material savings shall be resulted from using 56 or 91-day compression test results. Moreover, with later ages of test results used, other cementing materials can be incorporated into the concrete mixture which improves the durability of concrete in terms of heat generation in hydration and other aspects.

**9. What are the differences between shear slump and collapse slump in slump test? (T3)**

There are three types of slump that may occur in a slumps test, namely, true slump, shear slump and collapse slump.

True slump refers to general drop of the concrete mass evenly all around without disintegration.

Shear slump indicates that the concrete lacks cohesion. It may undergo segregation and bleeding and thus is undesirable for the durability of concrete [46].

Collapse slump indicates that concrete mix is too wet and the mix is regarded as harsh and lean.

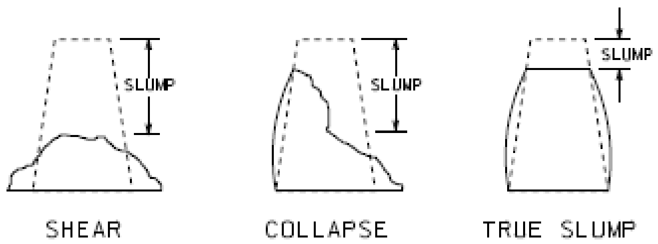


**Fig. Type of concrete slump**

**10. If on-site slump test fails, should engineers allow the contractor to continue the concreting works? (T3)**

This is a very classical question raised by many graduate engineers. In fact, there are two schools of thought regarding this issue.

The first school of thought is rather straightforward: the contractor fails to comply with contractual requirements and therefore as per G. C. C. Clause 54 (2)(c) the engineer could order suspension of the Works. Under the conditions of G. C. C. Clause 54(2)(a) – (d), the contractor is not entitled to any claims of cost which is the main concern for most engineers. This is the contractual power given to the Engineer in case of any failure in tests required by the contract, even though some engineers argue that slump tests are not as important as other tests like compression test.

The second school of thought is to let the contractor to continue their concreting works and later on request the contractor to prove that the finished works comply with other contractual requirements e.g. compression test. This is based upon the belief that workability is mainly required to achieve design concrete compression strength. In case the compression test also fails, the contractor should demolish and reconstruct the works accordingly. In fact, this is a rather passive way of treating construction works and is not recommended because of the following reasons:

(i) Workability of freshly placed concrete is related not only to strength but also to durability of concrete. Even if the future compression test passes, failing in slump test indicates that it may have adverse impact to durability of completed concrete structures.

(ii) In case the compression test fails, the contractor has to deploy extra time and resources to remove the work and reconstruct them once again and this slows down the progress of works significantly. Hence, in view of such likely probability of occurrence, why shouldn't the Engineer exercise his power to stop the contractor and save these extra time and cost?

# Level Two (Advanced FAQs)

## *Part I: Concrete Joint and Cracking Design*

### 1. Can a concrete structure be completely free of expansion joints and contraction joints? (J1)

Consider that the concrete structure is not subject to the problem of differential settlement.

For contraction joints, it may be possible to design a concrete structure without any contraction joints. By using sufficient steel reinforcement to spread evenly the crack width over the span length of the structure, it may achieve the requirement of minimum crack width and cause no adverse impact to the aesthetics of the structure. However, it follows that the amount of reinforcement required is higher than that when with sufficient contraction joints.

For expansion joints, the consequence of not providing such joints may be difficult to cater for. For example, a concrete structure has the coefficient of thermal expansion of $9\times10^{-6}$ /$^{o}$C and a Young's modulus of 34.5kN/mm$^{2}$. With an increase of temperature of 20$^{o}$C and it is restricted to free expansion, then the structure is subject to an axial stress of 6.21MPa. If the structure is very slender (e.g. concrete carriageway), buckling may occur. Therefore, the structure has to be designed to take up these thermal stresses if expansion joints are not provided. However, for water retaining structures, most of them are not affected by weather conditions because they are insulated from the water they contain internally and soil backfill that surround them. Therefore, it is expected that a smaller amount of thermal movement will occur when compared with normal exposed concrete structure. Consequently, expansion joints may be omitted in this case with the view that the compressive stress induced by thermal expansion toughens the structure to limit the development of tensile stress.

### 2. What are the functions of different components of a typical expansion joint? (J1)

In a typical expansion joint, it normally contains the following components: joint sealant, joint filler, dowel bar, PVC dowel sleeve, bond breaker tape and cradle bent.

Joint sealant: it seals the joint width and prevents water and dirt from entering the joint and causing dowel bar corrosion and unexpected joint stress resulting from restrained movement.

Joint filler: it is compressible so that the joint can expand freely without constraint. Someone may doubt that even without its presence, the joint can still expand freely. In fact, its presence is necessary because it serves the purpose of space occupation such that even if dirt and rubbish are intruded in the joint, there is no space left for their accommodation.

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

PVC dowel sleeve: It serves to facilitate the movement of dowel bar. On one side of the joint, the dowel bar is encased in concrete. On the other side, however, the PVC dowel sleeve is bonded directly to concrete so that movement of dowel bar can take place. One may notice that the detailing of normal expansion joints in Highways Standard Drawing is in such a way that part of PVC dowel sleeve is also extended to the other part of the joint where the dowel bar is directly adhered to concrete. In this case, it appears that this arrangement prevents the movement of joint. If this is the case, why should designers purposely put up such arrangement? In fact, the rationale behind this is to avoid water from getting into contact with dowel bar in case the joint sealant fails. As PVC is a flexible material, it only minutely hinders the movement of joint only under this design.

Bond breaker tape: As the majority of joint sealant is applied in liquid form during construction, the bond breaker tape helps to prevent flowing of sealant liquid inside the joint .

Cradle bar: It helps to uphold the dowel bar in position during construction.

**3. What is the purpose of using movement accommodation factor for joint sealant? (J2)**

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 [12]. Failure to comply with this requirement results in overstressing the joint sealants.

For instance, if the expected movement to be accommodated by a certain movement joint is 4mm, the minimum design joint width can be calculated as 4÷30% = 13.3mm when the movement accommodation factor is 30%. If the calculated joint width is too large, designers can either select another brand of joint sealants with higher movement accommodation factor or to redesign the arrangement and locations of joints.

### 4. What is the function of primers in joint sealant? (J2)

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 Constructional Sealants. When joint sealant is applied on top of joint filler in concrete joints, additional primers are sometimes necessary because [12]:

(i) Primers help to seal the surface to prevent chemical reaction with water;
(ii) It provides a suitable surface for adhesion of joint sealant.

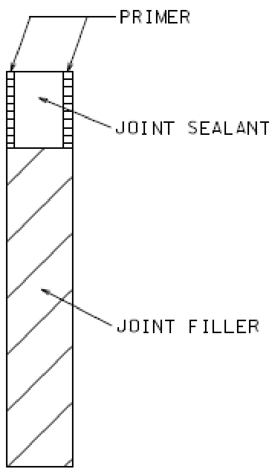


**Fig. Primer in joints.**

### 5. Is joint filler essential in concrete expansion joints? (J3)

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.

**6. What is the reason of using bond breaker for joint sealant? (J2)**

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. 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.

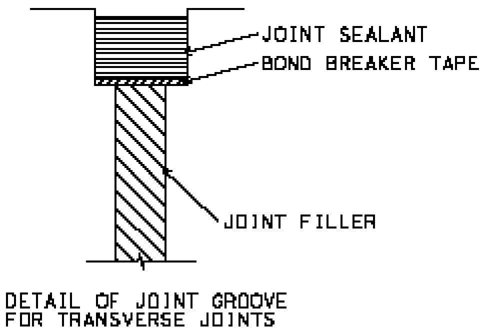


**Fig. Bond breaker tape for concrete joints.**

## 7. Can the depth exceed the width of joint sealant in concrete joints? (J2)

The shape of joint sealant affects its ability to stretch with movement. For instance, for rectangular joint sealant if the depth exceeds the width it tends to resist stretching of sealant in thermal movement. Moreover, block shape, when compared with concave shape, appears to be more resistant to stretching.

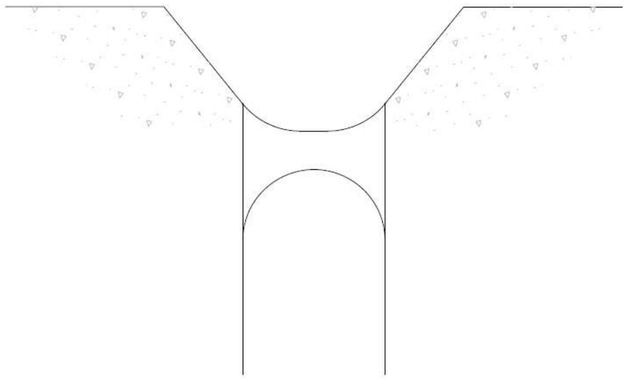

**Fig. Concave shape sealant**

## 8. Is joint filler essential in concrete expansion joints? (J3)

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**9. What is the significance of isolation joints? (J1)**

Isolation joints isolate slabs or concrete structure from other parts of structure. The presence of isolation joints allows independent vertical or horizontal movement between adjoining parts of the structure. Otherwise, the structure may experience cracking owing to the restrained movement caused by directional connection between adjoining concrete structures.

**10. Is waterstop necessary in construction joints of water-retaining structures? (J1)**

Construction joints are introduced during construction and they are not intended for movement to take place. If the concrete panel on each side of the construction joint has been designed to have maximum crack width of 0.2mm, theoretically this joint would also behave in the same way as its adjoining material. Hence, the design of this part of water-retaining structure such as concrete and steel are identical on both sides of panel. As such, BS8007 makes provision that it is not necessary to incorporate waterstops in properly constructed construction joints based on the above reasoning.

**11. What is the purpose of critical steel ratio in concrete structures?**

The purpose of critical steel ratio is to control the cracking pattern by having concrete failing in tension first. If steel reinforcement yields first before the limit of concrete tensile strength is reached, then wide and few cracks would be formed. In the calculation of critical steel ratio, the thickness of the whole concrete section is adopted for analysis. However, if the concrete section exceeds 500mm in thickness, only the outer 250mm concrete has to be considered in calculating minimum reinforcement to control thermal and shrinkage cracks [36]. It is because experimental works showed that for concrete section greater than 500mm, the outer 250mm on each face could be regarded as surface zone while the remaining could be regarded as core. The minimum reinforcement to control cracking should therefore be calculated based on a total maximum thickness of 500mm.

**12. What is the crack pattern induced by hydration due to internal restraint?**

Let’s take a circular column as an example to illustrate this.

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 formation of radial cracks near the surface of concrete.

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.

**13. Is the material of formwork (timber or steel) helps to reduce thermal cracks in concreting operation? (C1)**

To answer this question, one must fully understand the effect of formwork on the temperature of concreting structure. Without doubt, with better insulation of structure by timber formwork, the overall rise of temperature and hence the peak hydration temperature is also increased. However, for a well-insulated structure, the temperature gradient across concrete element is reduced. Therefore, the use of well-insulated formwork (like timber) increases the maximum temperature and reduces the temperature gradient across the structure at the same time. Hence, whether steel or timber formwork should be used to control thermal cracking is dependent on the restraints and the size of section.

If the section under consideration is thick and internal restraint is the likely cause to thermal cracking, then timber formwork should be used. On the other hand, if external restraint is the main concern for thermal cracking, then steel formwork should be used instead.

**14. For the purpose of defining the serviceability crack width limit state, the maximum design surface crack widths for the exposure conditions defined in BS8007 should be taken to be the following:**

> **The maximum design surface crack widths for direct tension and flexure <u>or</u> restrained temperature and moisture effects are:**
> **1) severe or very severe exposure: 0.2 mm;**
> **2) critical aesthetic appearance: 0.1 mm.**

**Is the crack width induced by concrete hydration and flexure should be considered individually to satisfy the above maximum crack width requirements? (C1)**

The crack width induced by concrete hydration and flexure should be considered individually to satisfy the above maximum crack width requirements. For crack width less than 0.2mm, it is assumed that the mechanism of autogenous healing will take place in which the crack will automatically seal up before the structure is brought into service. The crack width induced by hydration, which is checked to be less than 0.2mm, is sealed up first and then it will be subject to flexure for another checking on crack width.

**15. What is the purpose of reducing the seasonal and hydration temperature by one-half in the calculation of crack widths arising from thermal movement? (C1)**

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.

**16. Is the requirement of crack width limitation (<0.5mm) be effective in control ling reinforcement corrosion? (C1)**

In many standards and code of practice of many countries, the allowable size of crack width is normally limited to less than 0.5mm for reinforced concrete structure to enhance the durability of concrete. The limitation of crack width can serve the aesthetic reason on one hand and to achieve durability requirement by avoiding possible corrosion of steel reinforcement on the other hand. Regarding the latter objective, site surveys and experimental evidence do not seem to be in favor of the proposition. Beeby [6] showed that there was no correlation between surface crack width (<0.5mm) and durability of reinforced concrete structure. In practice, most corrosion problems are triggered by the presence of surface cracks parallel to the reinforcement instead of surface cracks perpendicular to the reinforcement.

**17. 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.

**18. What is the difference between plastic shrinkage cracks and crazing cracks? (C2)**

Plastic shrinkage cracks are caused by a rapid loss of water from concrete surface before setting of concrete such that the rate of evaporation of surface water is higher than the rate of replacement of upward rising water. Tensile force is developed at concrete surface which forms plastic shrinkage cracks when the concrete starts to stiffen. Plastic shrinkage cracks appear to be parallel to each other with spacing of about 300mm to 1m. The cracks are shallow and generally do not intersect the perimeter of concrete slab.

Crazing is the formation of a network of fine cracks on concrete surface caused by early shrinkage of surface layer. The pattern of crazing cracks is in the form of irregular hexagon. The cause of crazing cracks is the shrinkage of concentrated dense cement paste at concrete surface. A wet mix tends to depress the coarse aggregates and form a highly concentrated cement paste and fines on surface. Hence, the difference between plastic shrinkage cracks and crazing cracks lies in the fact that crazing cracks arise from the shrinkage of weak material such as laitance in concrete surface while plastic shrinkage cracks appear even in normal concrete surface.

**19. What are the reasons of occurrence of dusting and scaling of concrete surface? (C2)**

Dusting refers to the formation of loose powder arising from the disintegration of concrete surface. For dusty concrete surface, they can be easily scratched by nails. The cause of dusting can be related to finishing works carried out before completion of concrete bleeding. The working back of bleeding water to concrete surface produces a low strength layer with high water cement ratio. It may also arise owing to inadequate curing and inadequate protection of freshly placed concrete against rain, wind and snow.

Scaling is the occurrence of peeling of hardened concrete surface as a result of freezing and thawing effect. Scaling occurs in non-air-entrained concrete as air entrainment is normally adopted to protect concrete against freezing and thawing. Spaying of sodium chloride deicing salts is also another common cause of scaling.

**20. What is the effect of concrete placing temperature on early thermal movement?**

The rate of hydration of cement paste is related to the placing temperature of concrete. The rate of heat production is given by the empirical Rastrup function:

$$H = H_o \times 2^{r(T-T1)}$$

$H_o$ = Rate of heat production at a reference temperature
T = Temperature where rate of heat production H
T1 = Temperature where rate of heat production $H_o$

r = 0.084

An 12$^{o}$C increase in placing temperature doubles the rate of reaction of hydration. Hence, concrete placed at a higher temperature experiences a higher rise in temperature. For instance, concrete placed at 32$^{o}$C produces heat of hydration twice as fast when compared with concrete placing at 20$^{o}$C. Hence, high concrete placing temperature has significant effect to the problem of early thermal movement.

**21. What are the differences in method to seal moving cracks and non-moving cracks in concrete?**

In devising a suitable method to seal up cracks detected on concrete surface, it is of paramount importance to determine if further movement would be expected for the cracks. If the crack is not expected to move further, it is sufficient to brush cement grout into it. For wider cracks, other materials like latex-cement mixture may be considered for sealing the crack.

When further movement is expected for the crack, seals wider than the cracks are recommended to be applied over the crack in order to reduce the strain around it to an acceptable level. Moreover, it is desirable to apply the treatment when the cracks are widest so that the sealing material is not subject to further extension. Care should be taken to prevent bonding of sealing material with the bottom of the crack to ensure that only direct tension forces are experienced in the sealing material.

**22. What is the difference in functions between internal waterstop and external waterstop? (J4)**

External waterstops are applied externally on the structures and they proved to be effective when installed on the face with a net clamping pressure. For instance, external waterstops can be placed on the outer face of a basement to guard against water entry into the basement.

Internal waterstops are applied internally within the thickness of concrete and it is usually adopted when water pressure can act in both ways. They proved to be effective measures to guard against water flow in both directions but its success lies on the proper installation of waterstops inside the concrete structure. For instance, the installation of waterstops inside concrete slab encounters the problem of improper compaction of concrete around the waterstops.

**23. Shall reversible moisture movement be taken into account in estimating movement for movement joints?**

The size of concrete is affected by changes in atmospheric humidity: moisture causes expansion while drying causes shrinkage. Such moisture movement is reversible. This is totally different from drying shrinkage in which concrete slowly loses moisture during hardening, thus causing irreversible shrinkage.

In fact, the variation of humidity and the estimated reversible moisture movement is not significant (about 30%) and therefore, its contribution to movement does not justify for movement joints as suggested by MN Bussell & R Cather (1995).

# Level Two (Advanced FAQs)

## *Part II: Formwork and Curing*

### 1. What is the difference between release agent, form oils, form releaser, demoulding agent? (F1)

Release agent, form oils, form releaser and demoulding agent are materials for separating formwork from hardened concrete. Though they generally refer to the same meaning, there are slight differences among these terms.

| | |
|---|---|
| Release agent: | Materials that contain ingredients which are chemically combined with cement. |
| Form oils: | Diesel oils or other oil types. |
| Form releaser and demoulding agent: | General terms to describe materials which perform separation of forms from concrete. |

### 2. How can release agent help to separate formwork from concrete? (F1)

There are generally two main types of form releaser: barrier type or chemically active type.

For barrier type (e.g. form oil), it creates a barrier between the form and the fresh concrete. However, the quick evaporation of diesel oils affects clean air.

For chemically active type (e.g. release agent), an active ingredient (e.g. fatty acid) chemically combines with calcium (lime) in the fresh cement paste. This calcium/fatty acid product (grease or metallic soap) is stable and causes the formwork to release from the hardened concrete. It is this slippery, greasy, non-water soluble soap which allows the easy releasing of formwork from hardened concrete.

### 3. What are the potential problems of excessive application of form oils? (F1)

The problems of excessive application of form oils are:

(i) It stains the surface of hardened concrete.
(ii) Excess oils have nowhere to escape and find its way inside the cement paste and form holes subsequently. The oils bead up because of its incompatibility with water in chemical nature.
(iii) Higher cost is associated with increased usage of form oils.
(iv) In a relatively short time essentially most diesel oil evaporates so that it creates environmental problem.

**3. How can permeable formwork improve the quality of concrete?**

Permeable formwork serves as a filter that allows excess water and trapped air to escape from concrete surface. During compaction by vibrators, the fluid movement through permeable formwork drives out air and water, leaving behind a denser and stronger concrete. The movement of water results in a decrease in water in fresh concrete and fines cement particles from interior concrete shall be carried towards the formwork. Hence, it lowers the water cement ratio at concrete surface and enhances a higher strength near concrete surface.

In traditional formwork, during concrete compaction process the vibration tends to force water to the surface of concrete mass owing to the rearrangement of solid particles in concrete. As such, the concrete in concrete/formwork interface possess more water than the interior concrete. Consequently, the higher water cement ratio at concrete surface would lower the surface strength. Permeable formwork functions and solves this problem by allowing excess water to pass through.

**4. What is the relation of pouring rate and temperature with concrete pressure on formwork? (F2)**

Freshly placed concrete exerts pressure on formwork during the placing operation. It is influenced by the rate of placing and the air temperature. For instance, if the concrete pouring rate is too slow, setting of concrete starts to take place. As a result, the concrete at the bottom of the formwork sets prior to the placing of fresh concrete at the top and the maximum pressure will be reduced.

Temperature affects the rate of hydration of concrete. The higher the air temperature is, the higher will be the rate of hydration reaction. Consequently, fresh concrete tends to set at a faster rate. The pressure exerted on formwork decreases with an increase in temperature. For this reason, formwork is subjected to a higher pressure exerted by fresh concrete in winter than in summer.

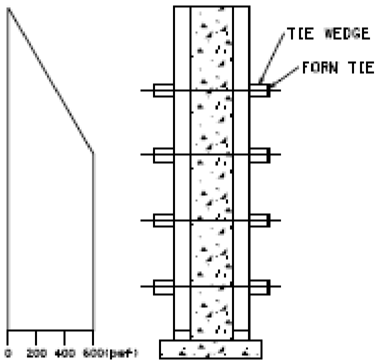



**Fig. Diagram of design concrete pressure diagram on formwork.**

**5. Is late removal of formwork beneficial to cater for early thermal movement? (F3)**

Let us take a circular column as an example to illustrate effect of internal restraint to thick sections.

When the temperature is rising, temperature in the core is higher than that at outer zone. The inner core will have a higher expansion and exert pressure to the outside. The induced compressive stress will result in the formation of radial cracks near the surface of concrete.

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.

It is beneficial for thick sections (say >500mm) to have late removal of formwork to reduce early thermal cracking. This is to allow more time for the centre of concrete section to cool down gradually to reduce the risk of thermal cracking. This is effective in controlling the temperature differential across the cross section of the concrete structures and reducing the potential of internal cracking due to early thermal movement.

**6. Comparing the rate of “Formwork exceeding 300mm wide, horizontal or at any inclination up to and including 5$^o$ to the horizontal” with the rate of “Formwork exceeding 300mm wide, at any**

**inclination more than 85$^o$ up to and including 90$^o$ to the horizontal", which one is higher?**

The item "Formwork exceeding 300mm wide, at any inclination more than 85$^o$ up to and including 90$^o$ to the horizontal" refers to formwork formed vertically and when compared with formwork erected in horizontal plane, the amount of falsework required is smaller.

The item "Formwork exceeding 300mm wide, horizontal or at any inclination up to and including 5$^o$ to the horizontal" refers to formwork to be erected in horizontal position and in general it requires much falsework to support this type of formwork. Therefore, the rate for this item is higher than the one mentioned in the above paragraph.

**7. In erection of falsework, for a rectangular panel inside a falsework should it be braced along the two diagonals? (F4)**

When a rectangular panel is subject to an eccentric load or a lateral load, it tends to deform into a parallelogram with one diagonal shortening and the other elongating. Theoretically, it is sufficient to brace along one of the diagonals (the one in tension). If one diagonal is only allowed to brace inside the rectangular panel, it should be not braced in the diagonal in compression because under severe lateral loading the diagonal may buckle leading to failure of structure.

However, in actual situation lateral loads may come from both sides of the panel and hence it should be braced in both diagonals.

**8. For long slender structures like beams, propping is required after removal of formwork. Why? (F4)**

After concreting, the time at which striking of formworks should not be too long, otherwise it would affect the colour of concreted structures. For long span concrete structures, when they have attained sufficient strength to support their self-weight, creep deflection may occur in these structures if propping is not provided after the removal of formwork. Therefore, re-propping is carried out after removing formwork and these props should not be allowed to stand too long because creep loads may overstress them.

Note: Propping refers to provision of falsework to support slabs and beams during their gain in concrete strength after concreting.

**9. Is curing compound suitable for all concrete? (C1)**

For concrete structures with low water-cement ratio (i.e. less than 0.4), it may not be suitable to use curing compound for curing. When hydration takes place, the relative humidity of interior concrete drops which leads to self-desiccation and drying-out. With no external supply of water, the cement paste can self-desiccate in such an extent that the hydration process stops. As such, curing compounds may not be sufficient to retain enough water in the concrete. In this case, wet curing is a better choice which serves to provide an external source of water.

**10. What is the difference between curing compound and sealing compound? (C1)**

Curing compound is primarily used for reducing the loss of moisture from freshly-placed concrete and it is applied once after concrete finishing is completed. Sealing compounds is adopted to retard the entrance of damaging materials into concrete and they are normally applied after the concrete is placed for 28 days. The harmful substances include water, deicing solutions and carbon dioxide which eventually cause freeze-thaw damage, steel corrosion and acid attack respectively.

**11. Curing time in summer is less than that in winter. Why?**

While concrete sets, it gains hardness and strength as the process of hydration slowly spreads the entire body of material. Curing should be allowed to continue for several days before subjecting the new concrete to significant stress. The period of curing depends on the temperature because the rate of all chemical reactions is dependent on temperature. Therefore, in summer the rate of reaction (hydration) is faster so that a shorter curing time is required. On the contrary, in winter the rate of reaction (hydration) is slower so that a longer curing time is required.

**12. Why does plastic sheet cause discolouration to freshly placed concrete? (C2)**

Plastic sheets are commonly used in curing to prevent moisture loss from concrete surface. However, it is not uncommon that discolouration occurs on the concrete surface. When plastic sheeting is spread over concrete surface and in direct contact with concrete, it tends to leave colour streaks on concrete surface. The problem of discolouration becomes even worse when calcium chloride is used in concrete mixes.

**13. What are the disadvantages of curing by ponding and polythene sheets? (C2)**

The purpose of curing is to reduce the rate of heat loss of freshly placed concrete to the atmosphere and to minimize the temperature gradient across concrete cross section. Moreover, curing serves to reduce of the loss water from freshly placed concrete to the atmosphere.

Ponding: This method of thermal curing is readily affected by weather condition (cold wind). Moreover, a large amount of water used has to be disposed off the construction sites after curing.

Polythene sheet: This method of curing is based on the principle that there is no flow of air over the concrete surface and thereby no evaporation can take place on top of the freshly concreted surface by provision of polythene sheets. However, it suffers from the demerit that polythene sheets can be easily blown off in windy condition and the performance of curing would be affected. Moreover, for water lost due to self-desiccation, this method cannot replenish these losses.

# Level Two (Advanced FAQs)

## *Part III: Steel Reinforcement*

### 1. What is the difference in application between open stirrups and closed stirrups in concrete beams?

Open stirrups are provided principally to resist shear forces in concrete beams and they are applied in locations in which the effect of torsion is insignificant. U-shaped stirrups are placed in the tension side of concrete beams in which shear cracks would occur. However, when concrete beams are designed to resist a substantial amount of torsion, closed stirrups should be used instead.

### 2. What are the reasons for establishing minimum distance between bars and maximum distance between bars? (SR1)

In some codes, a minimum distance between bars is specified to allow for sufficient space to accommodate internal vibrators during compaction.

On the other hand, the restriction of maximum bar spacing is mainly for controlling crack width [49]. For a given area of tension steel areas, the distribution of steel reinforcement affects the pattern of crack formation. It is preferable to have smaller bars at closer spacing rather than larger bars at larger spacing to be effective in controlling cracks. Hence, the limitation of bar spacing beyond a certain value (i.e. maximum distance between bars) aims at better control of crack widths.

### 3. What are the reasons of establishing minimum area of reinforcement and maximum area of reinforcement? (SR2)

Beams may be designed to be larger than required for strength consideration owing to aesthetics or other reasons. As such, the corresponding steel ratio is very low and the moment capacity of pure concrete section based on the modulus of rupture is higher than its ultimate moment of resistance. As a result, reinforcement yields first and extremely wide cracks will be formed. A minimum area of reinforcement is specified to avoid the formation of wide cracks [49].

On the other hand, a maximum area of reinforcement is specified to enable the placing and compaction of fresh concrete to take place easily.

**4. Why is tension anchorage length generally longer than compression anchorage length? (SR3)**

Tension anchorage length of steel reinforcement in concrete depends on bond strength. When steel reinforcement is anchored to concrete and is subjected to compressive forces, the resistance is provided by the bond strength between concrete and steel and the bearing pressure at the reinforcement end. Tension lap length is generally longer than compression lap length. In some design codes, instead of permitting the use of bearing pressure at reinforcement ends, the allowable ultimate bond stress is increased when calculating compression anchorage length.

**5. Why does lap length generally greater than anchorage length? (SR3)**

In some structural codes, the lap length of reinforcement is simplified to be a certain percentage (e.g. 25%) higher than the anchorage length. This requirement is to cater for stress concentrations at the end of lap bars. A smaller load when compared with the load to pull out an anchored bar in concrete triggers the splitting of concrete along the bar because of the effect of stress concentration. A higher value of lap length is adopted in design code to provide for this phenomenon.

**6. Does longitudinal steel serve as an enhancement of shear strength?**

In addition to shear resistance provided by shear reinforcement, shear forces in a concrete section is also resisted by concrete compression force (compressive forces enhances higher shear strength), dowel actions and aggregate interlocking. The presence of longitudinal steel contributes to the enhancement of shear strength of concrete section in the following ways [46]:

(i) The dowelling action performed by longitudinal reinforcement directly contributes significantly to the shear capacity.
(ii) The provision of longitudinal reinforcement also indirectly controls the crack widths of concrete section which consequently affects the degree of interlock between aggregates.

**7. Why are longer tension lap lengths designed at the corners and at the top of concrete structures? (SR3)**

In BS8110 for reinforced concrete design, it states that longer tension lap lengths have to be provided at the top of concrete members. The reason behind this is that the amount of compaction of the top of concrete members during concrete placing is more likely to be less than the remaining concrete sections [49]. Moreover, owing to the possible effect of segregation and bleeding, the upper layer of concrete section tends to be of lower strength when compared with other locations.

When the lap lengths are located at the corners of concrete members, the degree of confinement to the bars is considered to be less than that in other locations of concrete members. As such, by taking into account the smaller confinement which lead to lower bond strength, a factor of 1.4 (i.e. 40% longer) is applied to the calculated lap length.

**8. What is the purpose of setting minimum amount of longitudinal steel areas for columns? (SR2)**

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:

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

**9. Why does the presence of tension reinforcement lead to increasing deflection in concrete structures?**

In BS8110 a modification factor is applied to span/depth ratio to take into account the effect of tension reinforcement. In fact, deflection of concrete structure is affected by the stress and the amount of tension reinforcement.

To illustrate their relationship, let's consider the following equation relating to beam curvature:

Curvature = 1/r = e/(d-x)

where r = radius of curvature
e = tensile strain in tension reinforcement
d = effective depth
x = depth of neutral axis

Provided that the tensile strain in tension reinforcement remains constant, the curvature of concrete structure increases with the depth of neutral axis. It is observed that the depth of neutral axis rises with tension steel ratio. Therefore, the curvature of concrete section is directly proportional to the tension steel ratio. In addition, the larger value of the depth of neutral axis enhances increased area of concrete compression so that the effect of creep on deflection appears to become significant.

**10. For column reinforcements, why is helical reinforcement sometimes designed instead of normal links?**

The use of links for column design in Britain is very popular. However, in U.S.A. engineers tend to use helical reinforcement instead of normal links because helical reinforcement has the potential advantage of protecting columns/piles against seismic loads. Moreover, when the columns 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 columns as suggested by G. P. Manning (1924). In addition, it can take up a higher working load than normal link reinforcement.

For instance, helical reinforcement is adopted in the design of marine piles in Government piers.

Note: Helical reinforcement refers to shear reinforcement which is spiral in shapes.

**11. What is the effect of rusting on steel reinforcement? (SR4)**

The corrosion of steel reinforcement inside a concrete structure is undesirable in the following ways:

(i) The presence of rust impairs the bond strength of deformed reinforcement because corrosion occurs at the raised ribs and fills the gap between ribs, thus evening out the original deformed shape. In essence, the bond between concrete and deformed bars originates from the mechanical lock between the raised ribs and concrete. The reduction of mechanical locks by corrosion results in the decline in bond strength with concrete.
(ii) The presence of corrosion reduces the effective cross sectional area of the steel reinforcement. Hence, the available tensile capacity of steel reinforcement is reduced by a considerable reduction in the cross sectional area.
(iii) The corrosion products occupy about 3 times the original volume of steel from which it is formed. Such drastic increase in volume generates significant bursting forces in the vicinity of steel reinforcement. Consequently, cracks are formed along the steel reinforcement when the tensile strength of concrete is exceeded.

**12. Hong Kong General Specification for Civil Engineering Works (1992 Edition) Clause 15.09 specifies that tying wires for reinforcement adjacent to and above Class F4 and F5 finishes should be stainless steel wires. Why? (SR4)**

If plain steel tying wires are used for reinforcement adjacent to Class F4 and F5 finishes, it poses the problem of rust staining which may impair the appearance of exposed concrete surfaces. The rate of corrosion of plain steel tying wires is similar to normal steel reinforcement. However, for tying wires with very small diameter, upon long exposure it stands a high chance of rusting completely and these rust will stain the formwork and significantly affect the concrete finish. Therefore, stainless steel tying wires are specified for locations in the vicinity of high quality of finishes to avoid rust staining by corroded typing wires.

Note: Tying wires are wires used for fixing and connecting steel reinforcement bars.

**13. Which type of bar reinforcement is more corrosion resistant, epoxy-coated bars, stainless steel bars or galvanized bars? (SR4)**

Based on the experiment conducted by the Building Research Establishment, it was shown that the corrosion resistance of galvanized steel was the worst among the three types of bar reinforcement. For galvanized steel bars, corrosion started to occur when a certain chloride content in concrete (i.e. 0.4% by cement weight) was exceeded. However, for epoxy-coated bars, they extended the time taken for cracking to occur when compared with galvanized steel bars.

The best corrosion resistant reinforcement among all is stainless steel. In particular, austenitic stainless steel stayed uncorroded even there was chloride contamination in concrete in the experiment. Reference is made to K. W. J. Treadaway (1988).

**14. Does the presence of rust have adverse impact to the bond performance of bar reinforcement? (SR4)**

In fact, the presence of rust in bars may not have adverse impact to the bond performance and it depends on the types of bar reinforcement under consideration.

For plain round bars, the rust on bars improves the bond performance by the formation of rough surfaces which increases the friction between steel and concrete.

However, for deformed bars, the same theory cannot apply. The presence of rust impairs the bond strength because corrosion occurs at the raised ribs and subsequently fills the gap between ribs, thus evening out the original deformed shape. In essence, the bond between concrete and deformed bars originates from the mechanical lock between the raised ribs and concrete. On the contrary, the bond between concrete and plain round bars derives from the adhesion and interface friction. With such differences in mechanism in bonding, the behaviour of bond between deformed bars and plain round bars in the presence of rust varies. Reference is made to CIRIA Report 147.

**15. What is the difference in bonding performance to concrete between epoxy-coated bars and galvanized bars?**

Based on the findings of CEB Bulletin 211 [11], the bonding of galvanized bars to concrete is lower in early age owing to hydrogen release when zinc reacts with calcium hydroxide in concrete and the presence of hydrogen tend to reduce the bond strength between galvanized bars and concrete. However, bonding will increase with time until the full bond strength of ungalvanized bars is attained.

For epoxy-coated bars, there is a 20% decrease in bond strength for bars placed at the bottom of concrete sections while for bars placed on the top there is no major difference in bond compared with uncoated bars.

**16. What is the purpose of skin reinforcement for deep beams?**

In BS8110, it states that secondary reinforcement should be provided for beams exceeding 750mm deep at a distance measured 2/3 depth from the tension face. Experimental works revealed that at or close to mid-depth of deep beams, the maximum width of cracks arising from flexure may be about two to three times larger than the width of the same crack at the level of surface where the crack originally forms.

The presence of crack is undesirable from aesthetic point of view. Moreover, it poses potential corrosion problems to reinforcement of deep beams. To safeguard against these crack formation, skin reinforcement is designed on the sides of deep beams to limit the formation of flexural crack widths. Though the principal function of skin reinforcement is to control crack width, it may be employed for providing bending resistance of the section.

# 4. Module Three: Drainage and Sewage Works

## Objectives

| Element | Description | Objective No. |
|---|---|---|
| **Drains** | | |
| Design of Drains | Match soffit | DD1 |
| | Spigot pointing downstream direction | DD2 |
| | Embankment/Trench condition | DD3 |
| Element of Drains | Shape | ED1 |
| | Bedding | ED2 |
| | Trench | ED3 |
| | Concrete surround | ED4 |
| Drain Type | Flexible pipes | DT1 |
| | Precast concrete pipes | DT2 |
| **Box Culverts, Manholes and Catchpits** | | |
| Box Culvert | Geotextile filter | BC1 |
| | Joints | BC2 |
| | Single-cell/Multiple-cell | BC3 |
| | Inlet and Outlet | BC4 |
| Manholes | Components | M1 |
| | Manhole covers | M2 |
| | Spacing | M3 |
| **Channels** | | |
| Channels | Joints | C1 |
| | Freeboard | C2 |
| | Access ramp | C3 |
| | Stoplog | C4 |
| | Outlet | C5 |
| | Dams | C6 |
| | Stilling basin | C7 |
| | Flap valves | C8 |
| | Penstock | C9 |
| Riprap Channel | Construction method | RC1 |
| **Sewers** | | |
| Sewers | Precast concrete | S1 |
| | Vitrified clay pipe | S2 |
| | Sewer Manholes | S3 |
| Design of sewers | Sediment deposition | DS1 |
| | Pipe deflection | DS2 |

| **Element** | **Description** | **Objective No.** |
|---|---|---|
| Pumps | Wetwell/dry well | P1 |
| **Hydraulic Design** | | |
| Hydraulic Design | Rational Method | HD1 |
| | Colebrook White formula/ Manning's formula | HD2 |
| | Moody Diagram | HD3 |
| | Flow design | HD4 |
| | Best hydraulic section | HD5 |
| | Hydraulic jump | HD6 |
| | Cavitation | HD7 |
| | Manhole loss | HD8 |
| | Flooding | HD9 |
| Box Culvert | Critical slope | BC1 |
| | Tailwater level | BC2 |
| **Cracking and Tests** | | |
| Cracking | Reasons for occurrence | C1 |
| Tests | Air test/water test | T1 |
| | Water absorption test | T2 |
| | Sand replacement test | T3 |

# Level One (Core FAQs)

## *Part I: Drains*

### 1. Why is it preferable to design stormwater drains to match soffit? (DD1)

Stormwater drains collect stormwater in their corresponding catchment areas during rainstorm and convey the collected water through outlets to the sea. Therefore, in considering the hydraulic design of stormwater drains, other than normal drainage pipe capacity to be taken into consideration, one should check the backwater effect due to tidal condition at outlets if the drains are located quite close to the downstream end of outlets.

Stormwater drains are normally designed to match soffit to avoid surcharging by backwater effect or when the downstream pipes are running full. Normally pipe size increases from upstream to downstream. For the case of matching drain invert, when outlet pipes are fully surcharged by tidal effect of the sea or when the downstream pipes are fully filled with stormwater, pipe sections immediately upstream of the outlet are also surcharged too. However, for the case of matching pipe soffit, the immediate upstream sections of outlet pipes are not totally surcharged even though downstream pipes are running full. However, it is not always practical to maintain soffit for all pipelines because it requires sufficient drop to achieve this.

Moreover, the flow of stormwater is mainly by gravity in the design of stormwater drains. In case the drains are designed to match invert, then it stands a high probability that the flow in the upstream smaller pipes has to be discharged against a head.

Note: Matching soffit means that all pipelines are aligned continuously with respect to the pipelines' crown level.

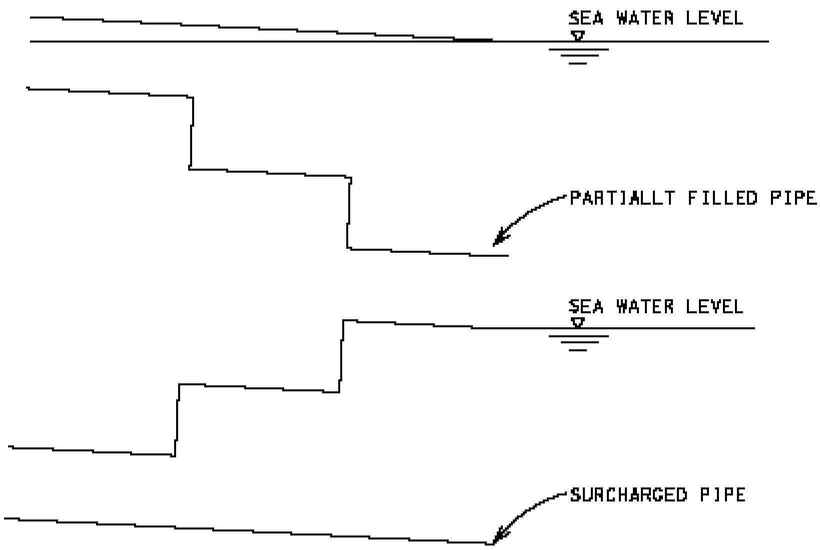



**Fig. Match soffit VS match invert**

### 2. Should precast concrete pipes be laid with spigot pointing downstream direction when fitted into sockets? (DD2)

There is a general rule of laying precast concrete pipes: *the precast concrete pipes should be laid from downstream end to upstream end*. Moreover, *precast concrete pipes should preferably be laid with spigot end pointing downstream direction when fitted into sockets*.

The reason of laying pipes from downstream to upstream is that any rainfall occurs during construction can be drained off the site without flooding the pipe trench. Moreover, gravity and other forces applied to the pipes during installation of pipe joints help tighten the joint and eliminate any remaining joint gap. On the other hand, for pipe laying with spigot point downstream direction, this prevents the joints from opening up arising from pipeline movement. The concrete pipe joints are also protected from the entry of foreign materials.

### 3. Which shape of drains is better, elliptical or circular? (ED1)

Horizontal elliptical pipes are commonly used where vertical clearance is hindered by some existing structures. Moreover, horizontal elliptical pipes possess higher flow capacity for the same flow depth than most other structures with equivalent full capacity.

For vertical elliptical pipes, it requires less excavation during trench installation owing to its narrow span. Moreover, backfill loads on the pipe is reduced when compared with circular pipes. Also, owing to its geometric shape, it is mostly used where there is limitation of horizontal clearance. From hydraulic point of view, vertical elliptical pipes allow higher self-cleansing velocity under dry season.

**4. Sometimes, "TOP' are observed on the surface of concrete pipes. What does it mean?**

It is obvious that pipes should lifted up and laid with "TOP" up. Otherwise, cracking may occur in the portion labeled "TOP" because they are not supposed to take up significant loads on the sides of the pipe.

In fact, the reinforcing cage in concrete pipes can be circular or elliptical. For pipes with elliptical reinforcing cage, it may contain the label "TOP" on its top. Elliptical reinforcing cage is more effective to resist loads because the reinforcement is mainly placed on the tension side of the concrete pipe. As such, the use of elliptical reinforcement needs less steel than circular ones for the same loading carrying capacity.

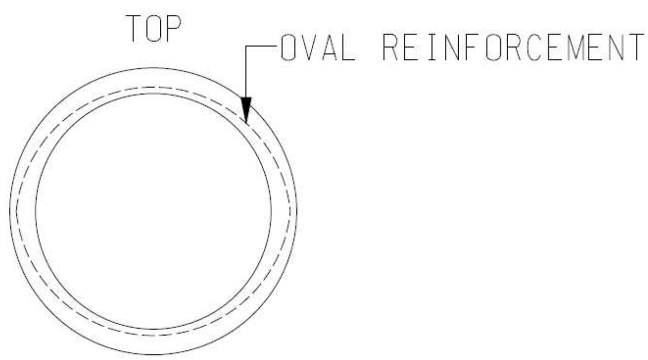



**Fig. Oval reinforcement in concrete pipes**

**5. What are the functions of bedding under stormwater drains? (ED2)**

Bedding, which are normally made of granular materials or concrete, serves four main functions as suggested by O. C. Young and J. J. Trott:

(i) To enhance a uniform support under pipes in order to reduce the bending moment longitudinally;
(ii) To increase the load-supporting strength of the pipes;

(iii) For pipes with spigot and socket joints, it enables pipes to be supported along pipe lengths instead of pipe sockets. Otherwise, uneven stress may be induced and it may damage the pipes;
(iv) To provide a platform for achieving correct alignment and level during and after construction.

## 6. What is the purpose of granular bedding for concrete pipes? (ED2)

In designing the bedding for concrete drainage pipes, granular materials are normally specified instead of soils containing a wide range of different particle sizes. The main reason of adopting granular material free of fine particles is the ease of compaction as it requires very little tamping effort to achieve a substantial amount of compaction and the crushed aggregates readily move to suitable place around the pipes [67]. However, the use of granular materials has the drawback that a stable support can hardly be provided for the drainage pipes. In particular, it cannot maintain an accurate slope and level for the bedding of concrete pipes. Most pipes are gravity pipes and the accuracy in level is essential to maintain the flow capacity.

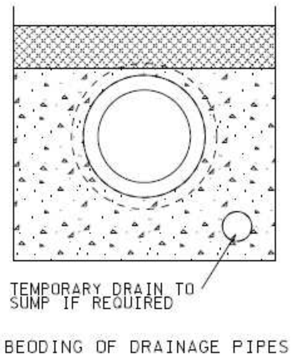



**Fig. Bedding of concrete pipes**

## 7. Should engineers consider embankment condition and trench condition when considering flexible pipes? (DD3)

The structural capacity of flexible pipes (e.g. plastics and metals) is derived from ring bending stiffness. Owing to creep or relaxation the ring bending stiffness decreases with time. Flexible pipes are liable to failure by excessive vertical deflections, ring bending strain and buckling.

For rigid pipes, the shape of embedment determines the amount of loads on pipes. For trench condition, the side walls of the trench provide frictional support to resist the filling material on top of pipes. For embankment condition, the fill materials on either sides of the pipe settle more than the soils on top of the pipe leading to increased loads on the pipe. However, for flexible pipes, it distort in the vertical direction at least as much as the embedment. The friction effects can hardly be developed to increase the loads on the pipes more than the loads on soil on its top. Hence, the shape of embedment is generally not considered when determining the loads on flexible pipes.

**8. Proper installation is essential for flexible pipes. Why? (DT1)**

Flexible pipes are pipes that can deflect at least 2% of the pipe diameter without any damage. When compared with rigid pipes such as precast concrete pipes, flexible pipes are comparatively weak and they count predominantly on the composite action between pipe deflection and backfilled soils to achieve structural stability. On the other hand, rigid pipes rely mainly on their inherent structural strength to carry imposed earth and traffic loads. Hence, improper installation of flexible pipes would compromise structural performance and results in risk failure.

The design of flexible pipes is complex as the soil boundary conditions include not only the backfilling trench but also the envelope of original soils outside the trench. It requires detailed assessment of theoretical pipe deflection, long-term bending strain and buckling pressure.

**9. What are the functions of different layers in the trench of flexible pipes? (DT1)**

A typical pipe trench for flexible pipes is divided into the following layers:

(i) Final backfill
This region has little influence on the performance of pipes. However, as it is close to existing road surface, it highly affects functioning of roads and structures.

(ii) Initial backfill
This zone provides some assistance in supporting pipe loads. It mainly serves to prevent the flexible pipe from damage upon placement of final backfill. It is beneficial to increase the depth of this region.

(iii) Haunching

This zone provides the resistance to pipe deflection and support pipe loads.

(iv) Bedding
This zone is commonly made up of compacted fill materials. It serves to provide even support for pipe laying and to maintain the pipe with correct line and level.

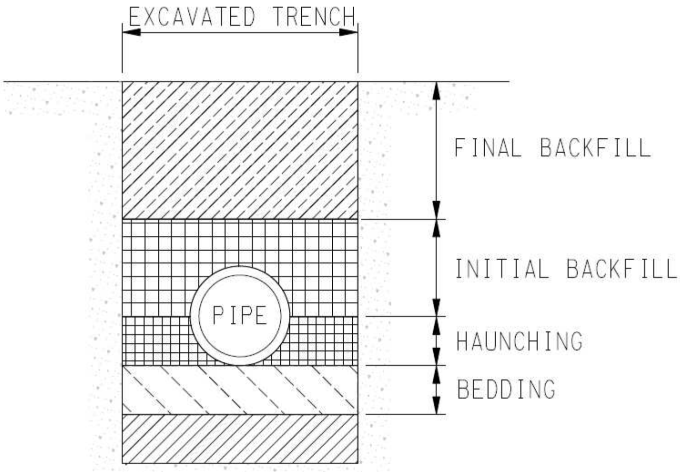



**Fig. Different bedding layers**

**10. Does sunlight impair the structural performance of PVC pipes? (DT1)**

Sunlight contains ultraviolet which transform PVC particles to a complex structures with brownish discoloration. The effect of discoloration can be reduced with addition of UV absorbers such as titanium dioxide.

Based on past research results, the impact resistance is affected by sunlight exposure for two years on PVC pipes while there is no effect on tensile strength and pipe modulus. As flexible pipes resist loadings by deformation of pipe, there shall be no impact on the pipe's load-carrying capacity owing to sunlight exposure. In general, smaller pipes with thin wall are more likely to be affected by sunlight than larger pipes with thick wall. For storage of PVC pipes for a long period, it is recommended to protect them against sunlight by tarpaulin or by painting. For painting PVC pipe, oil and solvent-based paint should not be used as this would dissolve the PVC pipe. Water-based paint should be adopted instead.

**11. What is the importance of trench width for the installation of flexible pipes? (DT1)**

Granular bedding is commonly used as bedding for flexible pipes. Granular material is self-compacting and it helps to reduce the pressure acting on the wall of pipe trench. Granular materials have higher modulus of soil reaction than adjacent in-situ soils and this allows it to carry higher loads than in-situ soils without deformation.

However, for poor in-situ soil conditions, the in-situ soils may not provide adequate lateral support when the flexible bends deform vertically and push the sides of pipe outwards the wall of pipe trench. As such, additional trench width is required so that thicker granular bedding on the sides of trench can be backfilled. As such, the thicker layer of granular bedding enhances the spreading of the force over a larger area so that the strength of poor soils is adequate to support it without failure.

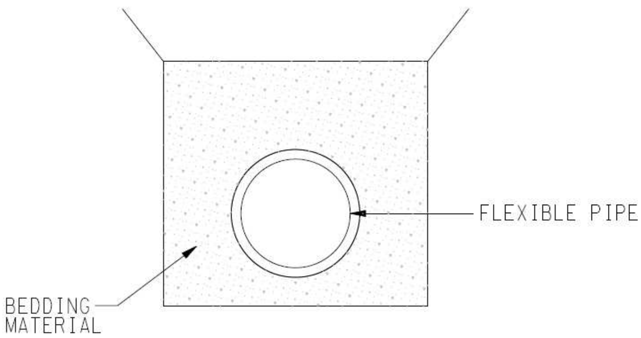



**Fig. Bedding material of flexible pipes**

**12. What is the relation of the angle of contact between pipe invert and bedding material to the load resisting capacity of pipe?**

Minimum crushing strength is a commonly adopted parameter for describing the strength of rigid pipes like concrete pipes. This value is determined in laboratory by subjecting the test concrete pipe to a line load diametrically along the pipe length while the pipe invert is supported on two bearers for stability reason. This test is called three-edge bearing test and the load at failure of pipes is expressed in terms of kN per length of test pipes (called minimum crushing strength).

Bedding factor of a pipe is defined as the failure load for the pipe laid in actual ground with bedding to the failure load under three-edge bearing test. The bedding factor is largely related to the angle of contact between pipe invert and the bedding material. The angle of contact between pipe invert and the bedding material increases with the ratio of bending moment at invert (for the case of three-edge bearing test) to the angle under consideration [67].

**13. What is the difference between narrow trench condition and embankment (wide trench) condition for drainage design? (DD3)**

In considering the loads on buried pipeline, there are normally two scenarios: narrow trench condition and embankment (wide trench) condition [23].

For narrow trench condition, when the pipe is laid in a relatively narrow trench with backfill properly compacted, the weight of fill is jointly supported by both the pipe and the frictional forces along the trench walls. For embankment condition, the fill directly above settles less than the fill on the side. Consequently, loads are transferred to the pipeline and the loads on pipeline are in excess of that due to the fill on pipeline.

The narrow trench condition is used where excavation commences from the natural ground surface without any fills above the surface. On the contrary, the embankment condition applies where the pipes are laid at the base of fill. For instance, embankment condition is normally adopted where the pipes are laid partly in trench or partly in fill or poor foundations to pipes are encountered so that the trenches have to be excavated wider than the minimum requirement.

**14. Why is it preferable to minimize the width of pipe trenches? (ED3)**

From the design point of view, it is preferred to minimize the width of pipe trenches because of the following reasons [29]:

(i) Higher cost of excavation is associated with wider pipe trenches.
(ii) The width of trench affects the loads on installed pipelines in consideration of embankment condition and wide trench condition. For minimum pipe trench width, the loads on pipelines can be reduced.

However, sufficient space has to be provided to allow for proper compaction. This is helpful to reduce the reaction at critical locations of pipelines under traffic and fill loads. Moreover, consideration should be given to accommodate temporary works for deep trenches where shoring has to be provided during construction.

**15. How does lateral pressure of soils affect the drain performance? (ED3)**

The presence of lateral pressure of backfilling sidesoils induces bending moments in the opposite direction from those produced by vertical loads and bedding support reaction. Such bending moment reduces the flexure in pipe wall and as a result this causes an equivalent increase in supporting strength of the pipe.

The lateral pressure of backfilling soils on drains is affected by the deflection of drains. With no occurrence of deflection, lateral pressure induced is in the form of active pressure. If pipe deflection occurs, the drain increases its horizontal dimension so that passive pressure is developed.

**16. Why are coupling system usually used for drains constructed in reclamation areas?**

Pipes used in drainage works are normally of spigot and socket type which is flexibly jointed. For this kind of pipe connections, it allows small amount of rotation and hence it could withstand certain degree of uneven ground settlement. Owing to uncertain nature of settlement in reclamation area where differential settlement is anticipated to occur, coupling system is usually provided for drains installed in reclamation area which provides a higher degree of resistance to differential settlement. Otherwise, it may cause cracking of pipelines leading to washing in of soil inside the pipelines. In the worst scenario, significant loss of soil may result in ground subsidence.

**17. What are the major areas to be paid attention when designing drainage pipes in reclamation areas?**

In reclamation areas drainage pipes are usually laid at flatter gradients when compared with upstream stormwater pipes. The fact that the nature of flow in stormwater drain is by gravity makes the downstream pipes in reclamation areas relatively deep below ground surface. It is preferable to have outfall of drains above the tidal influence level and this accounts for the relative flatter gradient of drain pipes in reclamation area.

Attention has to be paid to the possible occurrence of differential settlement in reclamation area. For pavement design, flexible pavement is preferred to rigid pavement to cater for settlement problems. Similarly, in the design of drains flexible joints like spigot and socket joints and movement joints in box culverts have to be provided to guard against the effect of differential settlement.

**18. Is it preferable to compact bedding for concrete pipes? (ED2)**

In the middle third of the base of precast concrete pipes, the bedding layers are recommended to be left uncompacted because it helps to reduce the reaction force at the invert of the pipes and intensifies the effect of shear forces. Moreover, the bending moment at pipe invert is increased by the compaction of bedding layer. The general rule for this region of bedding layer is that it should be firm enough for the pipes to rest on.

The sides of haunch and bedding directly under the haunch should be compacted because this will reduce the bending moment at the invert which is the critical failure location for pipes. The compacted haunch helps to resist the pipe load and maintain level and alignment.

**19. When branch pipelines are connected to main pipelines, sometimes Y-junctions or fitting branched pipelines to main pipelines by formation of holes in main pipelines are used. Which one is a better choice?**

By using standard precast units of Y-junction branch pipelines, it is beyond doubt that joints between branched pipelines and main pipelines are properly formed and the quality of joints is relatively less dependent on workmanship. However, it suffers from the problem that with fixed precast units of Y-junctions, sometimes it may be difficult for contractors to determine the precise orientation of specific angles of Y-junctions with respect to gullies. (e.g. gullies are connected through side branches to carrier drains)

By forming elliptical holes in main pipelines and fitting the side branches into them with cement mortar, the quality of pipe joints is highly dependent on workmanship. It is commonly found that in subsequent CCTV inspections side branches are projected inside main pipes. This is undesirable because the projected side branches reduce the cross sectional area of main pipes locally and affect their hydraulic performance. Moreover, the projected side pipes may trap rubbish and dirt in the vicinity. On the other hand, cement mortar may not be properly applied at connection joints because these areas are hidden from view and are difficult to be inspected by engineers. Therefore, in selecting between the two available methods, engineers should make their own judgments based on the above considerations.

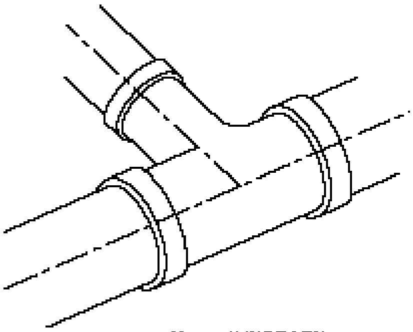



**Fig. Y-junction in drainage pipes.**

## 20. Should the distribution of reinforcement be designed as uniform for concrete surround of drainage pipes? (ED4)

Concrete surround is normally adopted for rigid drainage pipes to resist high traffic loads (e.g. under shallow covers) and to allow for using pipes with lower strength. Moreover, the use of concrete surround can minimize settlement of adjacent structures. In addition, the highest possible accuracy in levels and gradient can be achieved by using concrete surround as considerable settlement is expected in other types of beddings like granular bedding.

The distribution of reinforcement in concrete pipes may not be uniform owing to the occurrence of tensile stresses in different locations around the circumference of the pipes [67]. For instance, tensile stresses are highest at the inner face of pipes at invert and crown levels and at the outer face of the two sides of pipes. An elliptical cage may be designed in order to optimize the usage of steel reinforcement.

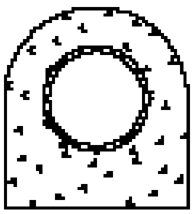



**Fig. Concrete surround of drains.**

## 21. What are the possible defaults for precast concrete pipes made by spinning and vertical casting? (DT2)

Small diameter precast concrete pipes are normally manufactured by spinning method. The spinning method basically makes use of the principle of centrifugal forces which diminishes towards the centre of precast pipe. Hence, problems like the presence of voids and variation of dimension occur frequently and remedial works like filling of voids by cement mortar has to be carried out depending on the severity of deficiency.

Large diameter precast concrete pipes are commonly produced by vertical casting method [67]. In this method the concrete pipes are normally placed upright with spigot staying on top, resting on socket moulds before the freshly-placed concrete has set. There is a possibility of deformation of pipe spigots to form oval shapes.

## 22. Does autogenous healing make the concrete pipe even stronger than original? (DT2)

Autogenous healing is common in underground drainage pipes because of the presence of water on either side of the pipes. The non-moving cracks in concrete pipes are sealed by calcium carbonate crystals from carbon dioxide in air and calcium hydroxide in concrete in the process called autogenous healing. The healed cracks are impermeable and behave even stronger than the original.

Autogenous healing can also be adopted to combat corrosion. The presence of water provides the basic conditions for both corrosion and autogenous healing. Corrosion shall take place with rusting of steel. At the same time, calcium carbonate from autogenous healing would be deposited on the cracks and the location of rusting. At last, the two processes compete i.e. disruption by volume expansion of rusting and the isolation of steel surface.

## 23. Why is it preferable to have backfilling partly-completed before carrying out water test? (ED3)

Water test is not intended to reassess the individual pipe performance because this should have been done in the manufacturing process. Instead, it serves to check the occurrence of faults during pipe laying process and to ensure that the pipeline can resist the internal hydrostatic service pressure it will be subjected to.

After adding water to the pipeline, 24 hours are usually specified before actual measurement. This stabilization time is required because both the escape of air trapped at pipe joints and water absorption take time to complete.

It is recommended that backfilling to pipes shall be partly-completed before the conducting of water test to cater for possible floating of pipelines in the event of accidental filling of pipe trench with water. Moreover, it is recommended to place adequate sidefill to restrain the pipeline to prevent movement of pipeline during water test. Otherwise, excessive movement of pipeline from water pressure may result in failure at joints.

**24. Why is air test considered for checking leakage in pipes though there is no direct relation between air loss and water leakage?**

There is no correlation between air loss and water leakage owing to the physical difference between air and water and the difference in behaviour of air and water under pressure conditions. Someone may doubt the philosophy in using air leakage to test for water leakage in pipes.

For cracks and openings of capillary tube size, surface tension may hinder the flow of water while air would still pass through the minute cracks. Based on past experience, for pipes passing air test is deemed to have satisfied water test (infiltration or exfiltration). For pipes which fail air test, it should also be subjected to water test before considering any replacement or rehabilitation.

## Level One (Core FAQs)

### ***Part II: Box Culverts, Manholes and Catchpits***

**1. Why is geotextile filter introduced below the rockfill layer of a typical box culvert? (BC1)**

In a typical box culvert, at the interface between rockfill layer and sub-grade, a layer of geotextile filter is usually added to perform separation function. With the addition of geotextile filter layer, it avoids the intermixing of widely different soil granulations so as to reduce long-term settlement. Moreover, it also prevents interpenetration of rockfill into sub-grade so that the deformed configuration of rockfill may impair its intended function of load spreading.

**2. Can dowel bars be omitted in the joints of box culvert? (BC2)**

Dowel bars in joint normally serve to maintain structural continuity by transmitting shear forces between adjacent concrete structures. For box culvert, the use of dowel bars in joints is essential owing to the following reasons:

(i) Without dowel bars, differential settlement would result and it leads to the formation of steps in the box culvert. As a result the flow capacity of box culvert would be reduced.
(ii) The steps in the box culvert would also provide locations to trap rubbish and debris.

**3. What is the function of waterstops in joints of box culverts and drainage channels? (BC2)**

The principal function of waterstops is to prevent liquids (e.g. water), water-borne materials and solids to pass through concrete joints. In essence, it aims at providing watertightness to the drainage channel.

Besides, waterstops in drainage channels or box culverts can also serve two other purposes: (i) to avoid water contacting joints' dowel bars and causing corrosion. (ii) to avoid water seeping in from the underside of drainage channels or box culverts, thereby washing in soil particles and causing voids underneath these structures and finally leading to their failure. To serve the second purpose, obviously only one waterstop is required at any depth location.

To serve the first purpose, a waterstop has to be installed on top of dowel bars to prevent water from drainage channels from leaking through. On the other hand, a waterstop has to be provided below dowel bars to avoid underground water from surging upwards.

In fact, the other way out to serve the first purpose is by using corrosion resistant bars.

**4. What is the difference between inlet control and outlet control in hydraulic design of box culvert? (BC4)**

In the hydraulic design of box culvert, there are two flow controls, namely inlet control and outlet control. In inlet control, the entrance characteristics of the box culvert (e.g. headwater depth and entrance configuration) determine the hydraulic capacity of the box culvert, and the culvert is actually capable of conveying a greater flow than the inlet would allow. Barrel shapes and tailwater depth are of no significance in determining the hydraulic capacity. Inlet control usually takes place for culverts lying on steep slopes.

For outlet control, the inlet could accept more flow than the box culvert could carry and the hydraulic capacity of the box culvert is dependent on all hydraulic factors upstream from outlet tailwater.

**5. Which one is better, single-cell box culvert and double-cell box culvert? (BC3)**

The use of double-cell box culverts is preferred to single-cell box culverts for cross-sectional area larger than about $5m^2$ owing to the following reasons:

(i) Where there is tight headroom requirement, the use of double-cell box culvert can shorten the height of culverts by having a wider base so that the same design flow can be accommodated.
(ii) The invert of one cell can be designed at a lower level to cater for low flow condition so that it reduces the occurrence of sediment deposition and avoid the presence of standing waters.
(iii) The provision of temporary flow diversion can be easily provided for inspection and maintenance of each cell. During routine maintenance operation, water flow can be diverted to one cell and the other one is

open for desilting.

If a choice has to be made between a single-cell box culvert and smaller multiple pipes, it is better to select single-cell box culvert because of the lower risk of blockage when compared with smaller size of multiple pipes. In addition, the hydraulic performance of a single-cell box culvert is better than multiple pipes system because of the larger hydraulic radius associated with the box culvert for a given cross-sectional area.

**6. What is the purpose in providing beveled edge in the inlet of box culvert? (BC4)**

The bevel is sometimes introduced at the inlet of box culvert to decrease the flow contraction at the inlet. In fact, the outlook of bevel is similar to a chamfer except that a chamfer is smaller in size and it is mainly used to prevent damage to sharp edges of concrete during construction. The bevels can be designed as plane or round edges.

The addition of bevels to the inlet of box culverts could increase the capacity of box culvert by about 5-10%. In fact, the socket end of precast concrete pipes serves the same function as the bevels in the box culvert.

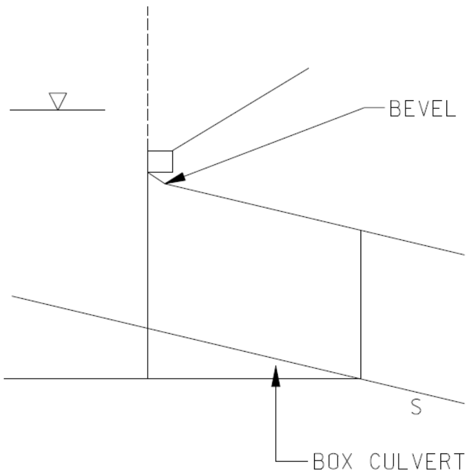



**Fig. Beveled edge in inlet**

**7. Should screens (trash or security) be always placed at culvert inlet? (BC4)**

Screens are provided at culvert inlet owing to one of the following reasons:

(i) To trap trash or debris which might otherwise accumulate in the

culvert and block the flow subsequently;
(ii) To prevent access into the culvert by children.

However, screens are not always placed at culvert inlet and they should be determined case by case. For trash or security screens installed in place, it would inevitably trap floating debris and rubbish. Unless they are regularly removed, it would eventually lead to rise in upstream water level leading to local flooding.

For new culvert, there is a trend that the use of trash screens is declining. Trash screens are only placed in culvert inlet where there is a high risk of blockage history. Instead the need of trash screen can be eliminated by adopting the following design features:

(i) There are fewer changes of cross section, fewer bends and smooth transitions into the culvert. As such, it would provide fewer locations to trap debris and trash.
(ii) Provision of good access to the culvert to facilitate regular inspection.
(iii) The culvert should be not designed too long to enhance easy access to clear a blockage.

**8. What are the differences in applications between pipe culverts and box culverts?**

Basically, a culvert means a covered hydraulic structure which conveys fluid. Therefore in a broad sense, pipe culverts in a small scale represent normal pipes like precast concrete pipes.

In terms of hydraulic performance, circular section is the best geometrical sections among all. Therefore, for relative small discharge, precast concrete pipes and ductile iron pipes are normally used which are circular in shape. But for applications of very large flow, precast concrete pipes and ductile iron pipes may not be available in current market. In this connection, cast-in-situ construction has to be employed. It is beyond doubt that the fabrication of formwork for circular shape is difficult when compared with normal box culvert structures. However, circular shape is the most hydraulic efficient structure which means for a given discharge, the area of flow is minimum. Therefore, it helps to save the cost of extra linings required for the choice of box culverts.

However, box culverts do possess some advantages. For example, they can cope with large flow situation where headroom is limited because the height of box culverts can be reduced while the size of pipe culverts is fixed. Secondly, for some difficult site conditions, e.g. excavation of structure in rock, for the same equivalent cross-sectional area, the width of box culverts can be designed to be smaller than that of pipe culverts and this enhances smaller amount of excavation and backfilling.

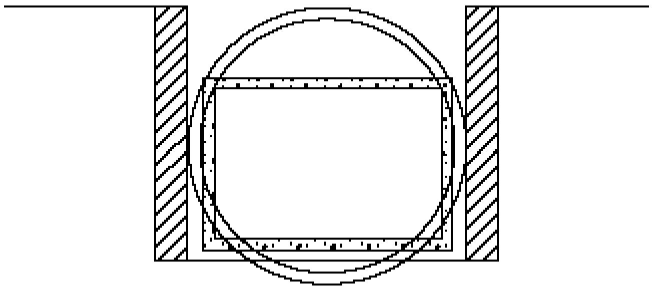


**Fig. Small spatial requirement of box culver than pipes.**

**9. What are the functions of the following features observed in a typical manhole? (i) groove near benching, (ii) R.S.J. (iii) double seal manhole cover and (iv) u-trap with rodding arm. (M1)**

(i) The groove is used to facilitate the maintenance of manholes and sewer/drain pipes. Shutoff boards are erected on the grooves during maintenance operation so that water flow coming from upstream is terminated in the manhole and backwater from downstream is also blocked. In addition, the groove also facilitates water flow diversion for routine maintenance operation.

(ii) R.S.J. is a small-scale size of universal beams and is used for resisting the high stresses incurred by heavy traffic loads acting directly on the upper narrow projected section of manholes.

(iii) Double seal terminal manhole covers are used for sealing off gases emitted inside sewer/drains and prevent them from releasing out of the manhole.

(iv) U-trap with rodding arms is also used for sealing off unpleasant gas smell by the trapped u-shaped water columns. Rodding arm is normally closed with rubber rings during normal operation. However, during maintenance operation, the rubber ring is removed and rodding can be carried out through the rodding arm.

## 10. How can manholes be adapted to the final height of the pavement? (M2)

Theoretically speaking, the whole manhole structures can be constructed as a whole instead of splitting into two stages of construction. With detailed calculation of longitudinal fall and cross fall of road pavement, it is possible to place and construct the manhole cover and frame in accurate levels and falls so that the whole manhole can be constructed in a single stage.

However, in actual practice it may not always be possible to accurately predict the inclination and levels of future manhole cover. As such, it is not uncommon that the construction of manholes is split into two stages:

(i) 1st Stage: The lower part of manholes is constructed up to the access shaft (about the level of sub-base). Then a rigid cover is placed over the access opening of the manholes and bituminous materials are placed on its top to construct the pavement.

(ii) 2nd Stage: The bituminous material around the rigid cover is removed and several courses of brickwork are normally placed on the access shaft to adjust the level of manhole cover and frame to fit with the final height of the pavement. Finally, manhole cover and frame are installed in position as shown in the sketch below.

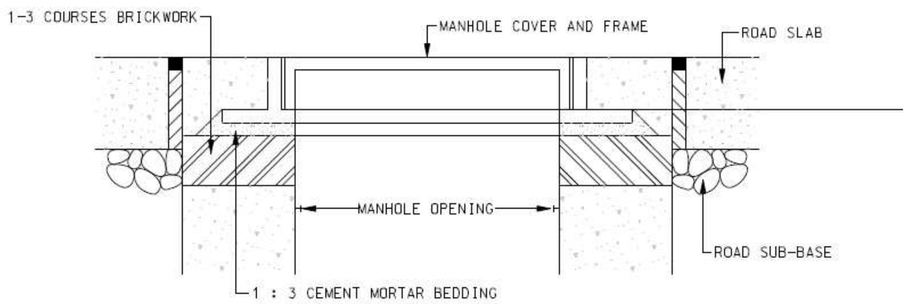



**Fig. Typical details of manhole cover and frame**

## 11. Why are some manhole covers designed into two triangular halves? (M2)

Manhole covers are generally made up of two pieces of triangular plates to form a square cover [23]. One may wonder why two rectangular halves are used for a rectangular cover. To understand this, one should note that a planar surface is usually in contact with three support points. For a triangular plate, the centre of gravity (CG) normally lies within the zone bounded by three support points even when loaded, thus creating a stable support system for the manhole cover. For a rectangular plate, the CG of a loaded manhole cover may lie outside the three support points, causing it to rock. Hence, the potential problem of rocking produced by vehicular traffic by rectangular traffic could be eliminated by using two triangular halves.

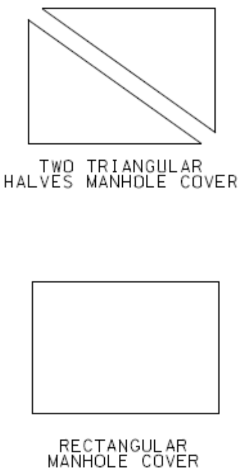



**Fig. Different types of manhole covers.**

**12. What are the potential problems of split triangular manhole covers? (M2)**

In some countries, manhole covers are designed into split triangular shapes to reduce the effect of rocking from traffic. However, the two pieces of triangular covers should be bolted together. As for a piece of triangular cover, it is easily dropped into the rectangular hole of manhole during routine maintenance. Therefore, from maintenance point of view, some countries prefer another geometrical shape i.e. circular, as this is the only shape that the cover could hardly be accidentally dropped into the manhole. On the other hand, for other geometrical shapes such as rectangle or square, they could still be dropped into their formed hole when inclined into proper angles.

**13. Why are some manhole covers made of cast iron while some are made of ductile iron? (M2)**

Traditionally, manholes covers are made of cast iron. However, in the viewpoint of pipe maintenance, frequent opening of manhole covers has to be carried out. Therefore, it poses potential safety hazard to the workers during the lifting-up process of manhole covers because cast iron manhole covers are very heavy to normal workers. Consequently, research has been conducted and ductile iron is considered as a better choice than cast iron because it can resist the same traffic loads with lower self-weight. Moreover, as ductile iron is less brittle than cast iron, the traditional cast iron manhole covers are more susceptible to damage and thus requires higher maintenance cost.

However, ductile iron manhole covers do suffer from some demerits. For instance, owing to their relative low self-weight, vehicles passing over these manhole covers would lead to the movement of covers and generate unpleasant noises. To solve this problem, instead of increasing the self-weight of ductile iron manhole covers which similarly causes safety problems to workers during regular maintenance, the covers can be designed to be attached to the manhole frames which hold them in firm position.

**14. The spacing of manholes in straight sections for different pipe sizes is stated in Stormwater Drainage Manual. How are these figures arrived at? (M3)**

For pipe size < 300mm, rodding is usually adopted in which workers place about 1m long rods through the pipes to the location of blockage and manually operate the rod to clear the blockage.

For pipe size < 700mm, water-jetting is normally employed in which water is supplied from nearby fire hydrants and pressurized water jet is used for clearing blockage.

Winching method is adopted for all sizes of pipes.

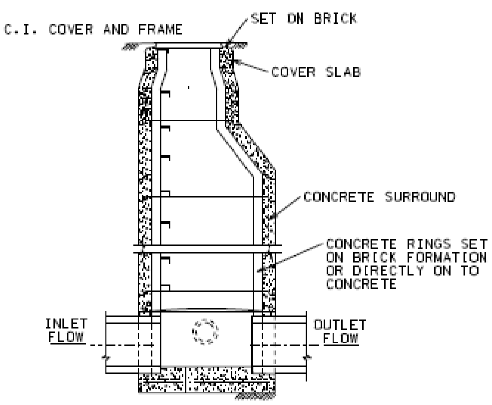
For instance, for pipe size exceeding 1050mm, it is stated in Stormwater Drainage Manual that maximum intervals between manholes along straight lengths should be 120m. This is because for sizes over 1050m, the main method of pipe maintenance is by winching whose maximum length of operation is 120m. Similarly, the maximum intervals of manholes for other straight pipes are derived from their corresponding maintenance methods.

**15. 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.

**16. Is reinforcement needed in precast concrete manhole units? (M1)**

Precast concrete manholes are normally constructed by placing the bases of manholes firstly. The walls of precast manholes are formed by placing the precast concrete rings one on top of the other up to the required height. Someone may notice that reinforcement used for resisting the lateral earth pressure and surface loads are not considered in some design. It is discussed in Concrete Pipe Association of Great Britain that analysis of soil pressures shows that standard unreinforced precast units are capable of resisting uniformly distributed pressures (e.g. loading condition in a manhole) down to a depth of 150m. If very severe road traffic and side loads are encountered, an additional concrete surround of about 150mm may be provided.



PRE-CAST CONCRETE MANHOLE

**Fig. Precast concrete manhole**

## 17. What is the difference between road gullies and catchpits?

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## Level One (Core FAQs)

### ***Part III: Channels***

**1. Two contraction joints and one expansion joints are usually adopted for drainage channels. Why? (C1)**

In the life cycle of a concrete structure (not prestressed concrete), it will generally undergo the following process of contraction and expansion:

Contraction: (a) Early thermal movement
(b) Seasonal contraction owing to drop in temperature
(c) Shrinkage
Expansion: (a) Seasonal expansion owing to drop in temperature

The order of magnitude for items (a) to (c) is more or less the same. Hence, qualitatively speaking, for a given length of concrete structure, the number of contraction joints should be more than the number of expansion joints and they are roughly in the order of 3:1 to 2:1 based on the number of expansion and contraction process above. Of course, the actual spacing and number of contraction joints and expansion joints should be determined case by case.

**2. Should joints in box culverts and channels be completely watertight? (C1)**

The joints for box culverts and channels should be capable of accommodating movements arising from temperature and moisture changes. However, the joints are not necessarily designed as watertight except the following conditions [15]:

(i) There is a high possibility of occurrence of high water table in the vicinity of box culverts/channels. 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.
(ii) If 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.
(iii) In cold countries, 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.

**3. Should the outer side of drainage channel at a bend be elevated to cater for superelevation?**

Flow around a bend results in a rise of water surface on the outside of the bend and it is natural to consider that extra height of channel wall on the outside of bend to prevent overflow of water.

However, for supercritical flow in channel, owing to the effect of superelevation extra height of channel wall should be provided on both sides of the bend. This is because supercritical flow around a bend would make water level go up alternatively on the outside and inside of the bend owing to cross waves. This cross wave pattern may continue for some distance downstream. As such, both sides of bend shall be lengthened to cater for this effect.

**4. Should the same freeboard be maintained along a channel? (C2)**

The freeboard is defined as the vertical distance from water surface to the top of channel bank. The selection of freeboard is dependent on the consequence should overflow out of channel bank occurs. Other than that, consideration should also be given to prevent waves, superelevation and fluctuations in water surface from overflowing the channel banks.

Generally, a 300mm freeboard is generally considered acceptable. For steep channels, it is preferably to have the height of freeboard equal to the flow depth to account for high variations in swift flow induced by waves, surges and splashes.

**5. In designing of access ramps for drainage channels, why should the direction of access ramps be sloping down towards downstream? (C3)**

In the design of access ramps, the direction is normally specified to be sloping down towards downstream so as to avoid the occurrence of over-shooting of flowing water for supercritical flow in case of aligning the ramps in the reverse direction of channel flow.

Note: Access ramps refer to ramps used for maintenance vehicles during routine maintenance of channels.

**6. What is the purpose of using riprap in drainage channels? (RC1)**

Riprap is an erosion-resistant ground cover made up of large, angular and loose stones (rock, concrete or other material) with geotextile or granular layer underneath. Riprap is commonly used in drainage channel to provide a stable lining to resist erosion by channel water. It is also used in channels where infiltration is intended but the velocity of flow is too large for vegetation.

A layer of geotextile is normally provided under riprap to perform separation from underlying soils. This prevents the migration of fined-grained soils from sub-grade into riprap and results in settlement and loss of ground.

**7. Should riprap be constructed by dumping or by hand-placing? (RC1)**

Riprap by dumping involves the dumping of graded stone by dragline or crane in such a way that segregation would not take place. Dumped riprap is a layer of loose stone so that individual stones independently adjust to shift in or out of the riprap. The dumped riprap is very flexible and would not be damaged or weakened by minor movement of the bank caused by settlement. Moreover, local damage or soil loss can be readily repaired by placement of more rock.

Riprap by hand-placing involves laying of stones by hand and by following a pattern with the voids between the large stones filled with smaller stones and the finished surface is kept even. The interlocking riprap produces a tidy appearance and decreases flow turbulence. Also, owing to the interlocking nature of riprap it allows the formation of riprap on steeper bank slopes. The thickness of riprap can usually be reduced when compared with dumped riprap. However, it requires much labour for installation of riprap. Another drawback is that the interlocking of individual rocks produces a less flexible revetment so that a small movement in the base material of the bank could cause failure of large portions of the revetment.

**8. Should angular or rounded stones be used in riprap channel? (RC1)**

Rock used for riprap should be blocky and angular, with sharp edges and flat faces. Angular stones proved to be effective to withstand external forces. Rounded stones have a high tendency to roll and inadequately protect the channel bed and bank. The ratio of length to thickness of angular stones should be less than 2.

If rounded stones have to be used, they should not be placed not steep embankments. Moreover, the size of rounded stones shall be increased (say 25%) with the corresponding increase in thickness of riprap layer.

**9. Why is stoplog seldom used in drainage channels? (C4)**

Stoplog consists of several sets of horizontal beams/logs stacked vertically. For narrow openings, the logs span between support slots at the ends of the openings. For wide openings, intermediate removable support posts may be required. They are prevalent in the past because of its low establishment cost, simple erection and easy operation.

However, there are also several drawbacks of this closure system. Since it requires a long lead time to mobilize workers and equipment for installation, it needs an accurate long-range weather forecast to allow for the long lead time. The situation is even worse for wide stoplogs which requires special lifting equipment for installation of intermediate support posts. When compared with time required to close sluice gates, the installation time of stoplogs is much longer. Moreover, a storage building has to be provided to prevent material loss by theft and damage by vandalism.

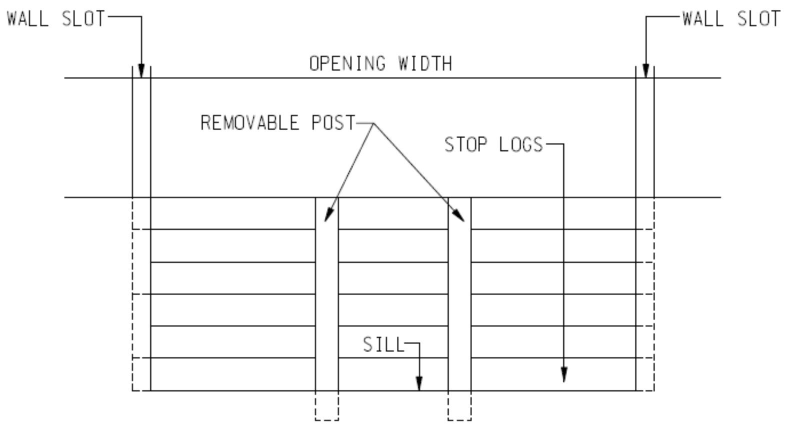



**Fig. Stoplog system**

## 10. Why is gabion apron necessary for gabion retaining wall to retain river embankment?

Gabion aprons are provided at gabion walls to protect its toe from scouring due to river flow. The scouring would eventually lead to undermining of the gabion structure and affects its structural stability. The length of gabion apron should be long enough such that it reaches beyond the limit where scouring may form. A layer of getextile filter is normally placed at the base of gabion structures to prevent leaching out of foundation soils.

## 11. How to cater for energy dissipation at drainage outlets? (C5)

Flow velocity at outlets is usually high. Without proper control of this energy, the subsequent bank erosion may result in failure of the banks. Therefore, some energy dissipating structures are designed to cope with this problem. Impact energy dissipaters may be provided at outlets by making use of impact walls to dissipate energy. Alternatively, the flows at outlet are dispersed artificially to achieve a significant loss of energy. However, the problem of cavitation may occur in this type of energy dissipating structures.

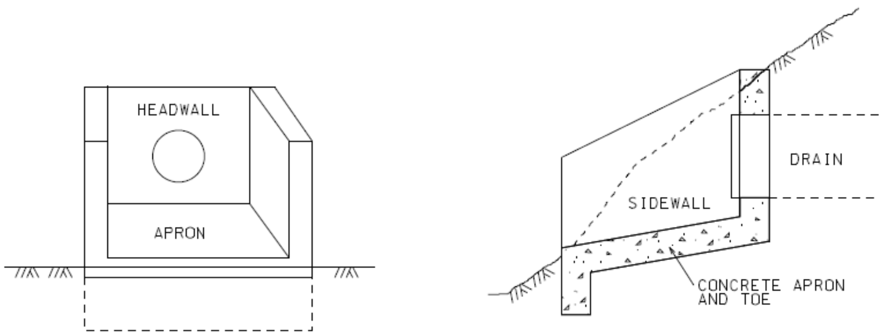



**Fig. A typical drainage outlet.**

## 12. In selection of dams in drainage channels, what are the advantages of using rubber dams instead of steel-gate dams? (C6)

The advantages of rubber dams are as follows:

(i) Since rubber is flexible in nature it is capable of performing deflation even in the presence of dirt and sedimentation on the downstream side. However, for rigid steel-gate dams, it may not be possible to open when there is excessive sediment.

(ii) Since the foundation of rubber dams is comparatively lighter than that

of steel-gate dams, it saves both construction cost and time.
(iii) Rubber dams can be designed with longer spans without piers while steel-gate dams require intermediate piers for long spans.

**13. How do we compare air-filled and water-filled rubber dams? (C6)**

Most of the existing rubber dams are of air-filled types. Water-filled rubber dams are not preferred for the following reasons:

(i) By giving the same sheet length and dam height, the tensile stress for water-filled dams is higher than that of air-filled rubber dams.
(ii) A significant size of water pond is normally provided for water-filled water dams for filling the rubber dams during the rising operation of dams.

**14. Under what conditions should engineers consider using stilling basins? (C7)**

Stilling basins are usually introduced to convert supercritical flow to subcritcal flow before it reaches downstream. A typical stilling basin consists of a short length of channels located at the source of supercritical flow (e.g. end of spillway). Certain features are introduced to the basins like baffles and sills to provide resistance to the flow. As such, a hydraulic jump will form in the basin without having conducting significant amount of excavation for the stilling basin if baffles are installed [31].

**15. When a drainage system (i.e. u-channels with catchpits) is connected to a main drainage channel, a segment of short pipe is used. What is the reason of such arrangement?**

There are three scenarios of such connection arrangement: (a) a new drainage system is connected to an existing drainage channel (b) an existing drainage system is connected to a new drainage channel (c) a new drainage system is connected to a new drainage channel.

For all scenarios, what engineers consider is the total amount of differential settlement or lateral movement to be encountered between the drainage system and main drainage channel. For scenario (b) and (c), it is very likely that substantial differential settlement will occur and this will cause damage to the connecting concrete pipes. Therefore a segment of short pipes are designed so that they serve to provide flexibility to the pipes in case of uneven settlement occurring between drainage system and main drainage channels.

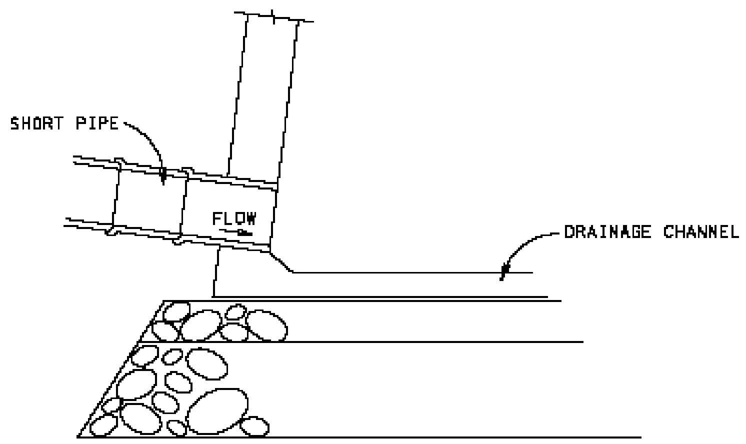



**Fig. Short pipe**

**16. Nowadays, most flap valves are made of HDPE. What are the advantages of using HDPE when compared with cast iron? (C8)**

(i) It has no reaction with sewage and seawater and does not suffer from the corrosion problem associated with cast iron.
(ii) No protective coating is required and it is almost maintenance-free.
(iii) HDPE flap valves require very low opening pressure in operation (like 5mm water level difference). For cast iron flap valves, due to its own heavy self-weight, the required opening pressure of cast iron flap valves is higher than that of HDPE flap valves. This criterion is essential for dry weather flow conditions.

However, the pressure resistance of HDPE flap valves is not as good as cast iron flap valves. For instance, a typical 450mm wide HDPE flap valve can only withstand about 5m water column.

**17. What is the difference between on-seating and off-seating head in penstock? (C9)**

A penstock is commonly used to control the flow and water level and for isolation of fluid. It mainly consists of a sliding door which is controlled by mechanical spindle moving through a hole in a frame built onto a structure. Penstock is the term used in UK while sluice gate is more commonly adopted outside UK. In the design of penstock, it is important to identify if it would take on-seating head or off-seating head.

On-seating head refers to the water pressure forcing the penstock into the wall while off-seating head refers to the water pressure forcing the penstock out of the wall as shown below.

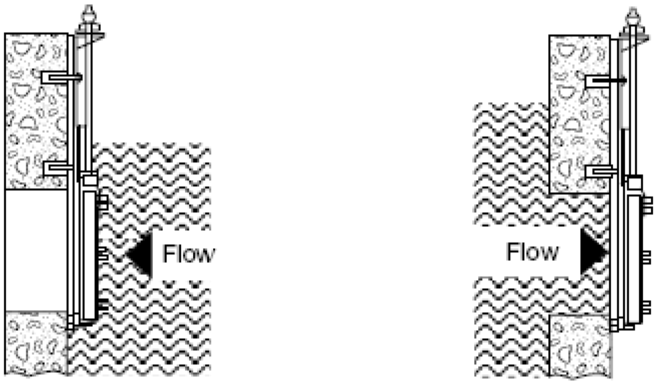



**Fig. On-seating (left) and off-seating head (right)**

# Level One (Core FAQs)

## *Part IV: Sewers*

**1. In designing sewer pipes, why are vitrified clay pipes commonly used for pipe size less than 600mm while precast concrete pipes with PVC lining is used for pipe size exceeding 600mm? (S1)**

The market price of vitrified clay pipes is generally less than that of precast concrete pipes with PVC lining. Therefore, for small size of pipes (pipe diameter less than 600mm) it is more economical to use vitrified clay pipes. However, vitrified clay pipes do suffer from the problem of brittleness and its effect is even severe for larger size of pipes. Moreover, it is rather time consuming to deliver clay pipes products because the majority of them are manufactured in Europe. Hence, for larger size of sewer pipes (diameter more than 600mm) it is customary to use precast concrete pipes with PVC lining.

**2. Why is partial PVC lining instead of full lining adopted in concrete pipes? (S1)**

The main function of PVC lining is to protect concrete surface against hydrogen sulfide attack. Moreover, it also guards against attack by a wide range of acids and alkalis. Hydrogen sulfide generated in sewerage by bacterial action under anaerobic (absence of oxygen) conditions is converted to sulphuric acid by aerobic bacteria growing on wet sewer walls. This acid reacts with the lime in the concrete, causing breakdown of the concrete pipes. The addition of lining could also improve the flow capacity.

Only the concrete above the line of minimum flow shall be attacked by hydrogen sulfide and therefore PVC lining is needed above this flow line. For medium to large pipe sizes, partial lining of 300 degree with 60 degree pipe invert exposed proves to be effective against attack. Small diameter pipe may warrant complete linings of pipes.

**3. Is vitrified clay pipes chemically resistant against all aggressive materials in sewage? (S2)**

In the manufacturing process of vitrified clay pipe, the clay material particles are fused into an inert and chemically stable compound. It is capable of carrying a wide range of commercial, industrial and domestic sewage. In fact, it is chemically resistant against sulfuric acid induced by hydrogen sulfide. Moreover, it is reported to be unaffected by the presence of solvents.

However, there are still some chemicals which are known to cause damage to vitrified clay pipe. For instance, vitrified clay pipe is not immune to attack by hydrofluoric acid and hot concentrated caustic wastes.

**4. Do extreme temperatures damage vitrified clay pipes? (S2)**

Vitrified clay pipe is capable of withstanding extreme temperatures. However, vitrified clay pipe is liable to damage by thermal shock – a swift change of temperature. Such rapid temperature variation induces thermal gradients inside the pipe wall which produces stresses which damage the pipe. To guard against possible damage of vitrified clay pipe by thermal shock, engineers should check in design about the temperature of sewage to be carried, rate of flow, temperature of pipe and soils and the wall thickness of clay pipes.

**5. What are the effects of sediment deposition in sewers? (DS1)**

There are mainly three major effects of sediment deposition in sewers:

(i) The deposited sediment may cause initiate small blockage of flow. Later when larger solid builds up it may result in total blockage of sewers.
(ii) The deposited sewer restricts flows leading to the drop in hydraulic capacity.
(iii) The deposited sediment serves as a point of pollutant storage or generator. The pollutants are stored temporarily and be discharged in high flow conditions. Moreover, biomedical change in sediment deposition releases gases which are corrosive to sewers.

**6. What are the limitations of CCTV regarding the inspection of sewers?**

CCTV is a well-established and prevalent method for pipe inspection. However, there are several limitations of CCTV technology:

(i) It could only provide the view of pipe internal surface above waterline.
(ii) It does not provide the view of soil conditions surrounding the pipe.
(iii) Most CCTV system is incapable of measuring pipe gradient.

(iv) It does not provide any structural data on the integrity of pipe wall.

**7. What is the application of inverted siphons? What are the disadvantages of using inverted siphons?**

Inverted siphons are designed at locations in which a sewer system is blocked by underground utilities or stormwater drains. They are sometimes called depressed sewers because it is claimed that there is no actual siphon action. They connect the upstream and downstream sewers with U-shaped vertical alignment such that they are always running full.

The drawbacks of inverted siphons are:
(i) They induce additional head loss to the sewer system which is undesirable in hydraulic performance;
(ii) U-shaped siphons create sediment accumulation problem and previous experience showed that inverted siphons were easily blocked due to siltation;
(iii) Maintenance of invert siphons is difficult due to its inaccessibility.

**8. Does pipe deflection affect its flow capacity? (DS1)**

When excessive pipe deformation occurs, it may impair the joint performance and affect the strain in pipes. Based on the information by PIPA, there is a 5% reduction of flow capacity when the pipe is deflected by 15%. Hence, pipe deflection has impact on flow capacity but its effect is not significant.

**9. What is the difference of movement of solids in large sewers and small sewers? (DS1)**

For solids in large sewers, forces on solids position them at different flow heights depending on their specific gravity.

There are generally two modes of movement of solids in small sewers, namely, *floating and sliding dam*. The floating mechanism operates when the size of solid is small when compared with the diameter of sewer. Solids move with the wave of sewage. On the other hand, the sliding mechanism functions when the size of solid is large when compared with the diameter of sewer. The sewage waves build up behind the solids which act as a barrier at the base of sewer. When the waves store sufficient energy to overcome the friction between solids and sewer invert, the solid would move along the sewer.

**10. What are the effects of sewer sediment on hydraulic performance? (DS1)**

The presence of sediment in sewers has adverse effects on the hydraulic performance of sewers [13]. For the case of sewage flow carrying sediment without deposition, the presence of sediment in the flow causes a small increase in energy loss.

In case the sewer invert already contains a bed of sediment deposit, it reduces the cross sectional area of sewers and consequently for a given discharge the velocity increases. As such, the head losses associated with this velocity increase. Moreover, the increase in bed resistance induced by the rough nature of sediment deposit reduces the pipe flow capacity of sewers.

For sewers which are partially full, the presence of sediment bed enhances higher frictional resistance and results in increasing the flow depths and subsequent decrease of velocity. The reduction of velocity will lead to further deposition of sediment owing to the decrease of sediment carrying capacity if the increase of capacity of sewers generated by the presence of sediment bed does not exceed the reduction in flow caused by the bed roughness.

**11. What are the functions of wetwells? (P1)**

Wetwells are designed to store temporarily water/sewage before it is pumped out. They are usually provided for sewage and stormwater pumping stations and they serve the following functions:

(i) They assist in attenuating the fluctuations of flow owing to the diurnal variation of sewage discharge.
(ii) The wetwells serve as sump pits where the suction pipes are inserted and the fluid level in the sumps can be employed for the control of opening and closure of pumps.

**12. Should sewer manholes be designed as watertight? (S3)**

Sewer manholes should preferably be designed to be watertight owing to the following reasons:

(i) Water-tightness is important in sewer system. Otherwise, leakage of sewage from manholes would seriously contaminate the environment.

(ii) In the event of high water table, water would infiltrate into sewer manholes. Therefore, a higher volume of sewage would be delivered to sewage treatment plant for treatment and this essentially increases the cost for treating this additional volume of water.

**13. What are possible causes of manhole explosions? (S3)**

It is not uncommon that manhole explosion occurs nowadays. Manhole covers are dislodged from the frames which is associated with a release of energy. Manhole explosion occurs mostly owing to the ignition and combustion of flammable gas. Sources of flammable gas include the followings:

(i) Natural gas as a result of leakage of nearby gas line;
(ii) In sewer manholes, it is rich in methane which tends to accumulate inside manhole;
(iii) Gas generated by degradation of cable insulation.

**14. In the design of dry well pumping stations, which arrangement is better, "turned-down" bellmouth or horizontal intake? (P1)**

Pumps can be installed as dry well or wet well. The wet well is commonly used because of its simplicity and low cost. However, this type of pump arrangement has the potential problem of maintenance. For instance, it requires the de-watering of sump and removal of pumps out of the sump, which is suitable for stormwater pumping station which does not require pumping for most of the time. For dry well, the pumps could be assessed and maintained all of the time.

In general "turned-down" bellmouth of pump inlet is more popular because of the following two reasons:

(i) It is less susceptible to vortex action with similar water height.
(ii) It accommodates a lower water cover.

**15. What is the purpose of bellmouth entry to a circular pipe for pumps? (P1)**

If sharp edged inlet to pipes connecting to a pump is adopted, flow separation will occur. Flow separates from sharp edges and a recirculation zone is formed. Moreover, turbulence shall form at downstream when the flow at vena contracta subsequently expands to fill up the unfilled void. Flow separation leads to significant head loss.

The design of bellmouth entry is to ensure that the flow is uniform over the entire intake section and the head loss induced at inlet section is minimized.

# Level Two (Advanced FAQs)

## *Part I: Hydraulic Design*

### 1. What are the limitations of Rational Method in calculating runoff? (HD1)

Computation of runoff is a complicated matter which depends on many factors like the ground permeability, rainfall duration, rainfall pattern, catchment area characteristics etc. Basically, Rational Method is a means to find out the maximum discharge suitable for design purpose. In this method, it is assumed that the rainfall duration is the same as the time of concentration and the return period of rainfall intensity is the same as the peak runoff. Time of concentration refers to the time required for the most remote location of stormwater inside the catchment to flow to the outlet. When the time of concentration is equal to the rainfall period, the maximum discharge occurs and rainfall collected inside the catchment comes to the same outlet point.

Rational Method provides the peak discharge only and it cannot produce a hydrograph. If a more detailed pattern of runoff is required, unit hydrograph or other methods have to be used. The accuracy of rational method depends very much on our correct selection of runoff coefficient and delineation of catchment area.

Rational Method is a rather conservative method. One of the basic assumptions of the rational formula is that the rainfall intensity must be constant for an interval at least equal to the time of concentration. For long duration of rainfall, this assumption may not hold true. Moreover, the runoff coefficient in Rational Method is difficult to be determined accurately and it depends on many factors like moisture condition of soils, rainfall intensity and duration, degree of soil compaction, vegetation etc. In addition, In Rational Method the runoff coefficient is independent of rainfall intensity and this does not reflect the actual situation.

### 2. Rational Method should not be used for large catchments in estimating peak runoff. Is it true? (HD1)

Rational Method is suitable for small catchments only because the time of concentration of small catchments is small. In Rational Method the peak runoff is calculated based on the assumption that the time of concentration is equal to the rainfall duration. For small catchments, this assumption may hold true in most circumstances. One of the assumptions of Rational Method is that rainfall intensity over the entire catchment remains constant during the storm duration. However, in case of a large catchment it stands a high probability that rainfall intensity varies in various part of the large catchment. In addition, for long duration of rainfall, it is rare that the rainfall intensity remains constant over the entire rainstorm and a shorter duration but a more intense rainfall could produce a higher peak runoff. Moreover, a reduction of peak runoff is also brought about by the temporary storage of stormwater like channels within the catchment.

In actual condition, the runoff rate within the catchment varies from place to place because of different soil properties and past conditions. As suggested by Bureau of Public Roads (1965), sometimes the peak discharge occurs before all of the drainage area is contributing. For instance, when a significant portion of drainage area within the catchment has very small time of concentration so that a higher rainfall intensity can be used for this portion, the runoff coming solely from this portion is higher than that of the whole catchment in which a lower rainfall intensity is adopted because the remaining part of the catchment has comparatively large time of concentration. Therefore, this results in incorrect estimation of peak runoff of large catchments if Rational Method is adopted.

**3. Is Colebrook White formula suitable for shallow gradient of pipes? (HD2)**

Manning's Equation is commonly used for rough turbulent flow while Colebrook-White Equation is adopted for transition between rough and smooth turbulent flow.

For Manning's Equation, it is simple to use and has proven to have acceptable degree of accuracy and is the most commonly used flow formula in the world. When using Colebrook-White Equation, it is observed that for very flat gradient (i.e. <1.5%) it tends to underestimate the flow because as gradient approaches zero, velocity also approaches zero. Hence, care should be taken when using Colebrook-White Equation for flat gradients.

**4. Why is Manning's formula more often used than Chezy formula in open channel flows? (HD2)**

Manning's formula was proposed by Robert Manning (an Irish engineer) to calculate uniform flow in open channel. It is probably the most widely used uniform-flow formula around the world. Its extensive usage is due to the following reasons:

(i) The majority of open channel flows lies in rough turbulent region;
(ii) It is simple in form and the formula is well proven by much practical experience.

Manning's formula shall not be applied in situations where Reynolds number effect is predominant. For Chezy formula, it is less commonly used owing to insufficient information to find out equivalent roughness and it is not backed by sufficient experimental and field data. However, in smooth boundaries, Chezy formula shall be adopted instead of Manning's formula.

**5. Does Moody Diagram used for calculating energy losses in pipes suitable for all conditions? (HD3)**

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%.

**6. Does full bore flow means maximum discharge in drainage design? (HD4)**

In the design of gravity drainage pipes, full bore flow capacity is normally adopted to check against the design runoff. However, one should note that the maximum flow rate does not occur under full bore conditions. The maximum discharge occurs when the water depth in circular pipes reaches 93.8% of the pipe diameter. Therefore, the use of full bore discharge is on the conservative side though the pipe's maximum capacity is not utilized.

Similarly, the maximum velocity does not occur in full bore conditions and for circular pipes it occurs when the water depth is 81.3% of the pipe diameter. Hence, in checking for the maximum velocity of flow in pipes to avoid possible erosion by rapid flow, the use of full-bore velocity may not be on the conservative side.

**7. What is the reason in checking the ratio (i.e. design flow to full-bore flow > 0.5) in circular pipe design? (HD4)**

For checking of self-cleansing velocity for pipes, there is another criterion to check design flow Q to full bore flow $Q_{full}$> 0.5. If this criterion is met, it can be deduced that the design flow is always greater than self-cleansing velocity.

The reason behind is that from the chart of circular pipes, when Q/ $Q_{full}$ >0.5, then the ratio of design velocity V to full bore velocity $V_{full}$ >1. After confirming $V_{full}$ >1m/s, then it leads to V>1m/s. Hence, minimum velocity at full bore flow should be checked.

**8. What are the potential advantages in using best hydraulic section? (HD5)**

The best hydraulic section of an open channel is characterized by provision of maximum discharge with a given cross sectional area. As such, channels with circular shape is the best hydraulic sections while a rectangular channel with channel width being equal to two times the height of channel is the best hydraulic section among all rectangular sections. In fact, the choice of best hydraulic section also possesses other advantages than hydraulic performance. For instance, for a given discharge rate the use of best hydraulic section could guarantee the least cross sectional area of the channels. Substantial savings could be made from the reduction in the amount of excavation and from the use of less channel linings.

**9. What are the potential problems of high velocity in pipes? (HD4)**

Flow velocity seldom causes abrasion problem for concrete pipes. Instead, the particles carried by effluent in high velocity may create abrasion problem of concrete. The abrasive effect is dependent on the size of particles and velocity. In most circumstances, the problem of abrasion shall be avoided for flow velocities less than 8m/s.

Very high velocity (i.e. more than 10m/s) could also induce significant cavitation problem because air bubbles are formed from low water pressure and they would collapse when entering a region of high water pressure.

**10. Should a box culvert be designed as free flow or surcharged flow? (HD4)**

Whenever possible box culverts should be designed for free flow at design flow rate. A box culvert with surcharged flow is similar to inverted siphon which creates maintenance problem. There is a risk of blockage by silt and debris and the inspection and maintenance of submerged culvert is difficult when compared with free flow condition. For long box culverts (more than 20m) it is even more important to design for free flow to decrease the risk of blockage which results in an inaccessible path.

From hydraulic point of view, the change from free flow to submerged condition would cause an increase in head loss leading to a rise in upstream water level.

**11. What are the potential problems of channels to carry supercritical flow? (HD4)**

Supercritical flow involves shallow water flowing in high velocity. The shallow water depth results in higher velocity head when compared with subcritical flow. The fast flow of water causes erosion to channel linings and beddings. When the channel slope becomes flat, the flow can become subcritical causing the formation of hydraulic jump which further causes erosion to channel bed.

Owing to swift flow of water in channels accommodating supercritical flow, there is considerable safety risk in the event of passengers falling into the channel and washed away to downstream. Moreover, it poses difficulty in carrying out maintenance operation of the channel owing to its high water flow.

**12. What is the significance of tailwater level in culverts? (BC2)**

The headwater level and tailwater level of culverts are important parameters in hydraulic design. The headwater level cannot be set too large, otherwise flooding upstream may occur leading to the loss of life and properties. On the other hand, the tailwater level of culverts has to comply with the following requirements [29]:

(i) For low tailwater levels at the outlet of culverts, the small depths of flow may cause significant erosion of downstream channels.
(ii) For high tailwater levels, it may cause the culvert upstream to be

flowing full or even under submerged condition. As such, the headwater level is increased in order to flow through the culvert and this in turn increases the flooding risk associated with high headwater level.

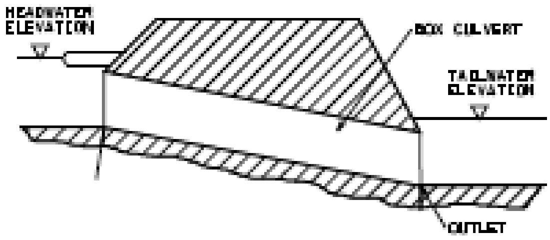



**Fig. Tailwater level in culvert.**

### 13. What is the significance of critical slope in hydraulic design of box culvert? (BC1)

Critical slope is the minimum slope in which maximum discharge shall occur without requiring the box culvert to flow full. For box culverts with slope less than critical slope, at low headwater it tends to flow full and eventually requires a higher headwater depth to convey the same amount of water required for culverts with slopes greater than critical slope.

### 14. What is the difference between on-line storage and off-line storage in the design of storage pond? (HD9)

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.

### 15. What are the effects of hydraulic jump? (HD6)

The use of hydraulic jump in hydraulic engineering is not uncommon and the creation of such jumps has several purposes [58]:

(i) Its main aim is to perform as an energy-dissipating device to reduce the excess energy of water flows.
(ii) The jump generates significant disturbances in the form of eddies and reverse flow rollers to facilitate mixing of chemicals.
(iii) During the jump formation, considerable amount of air is entrained so that it helps in the aeration of streams which is polluted by bio-degradable wastes.
(iv) It enables efficient operation of flow measuring device like flumes.

**16. What is the mechanism of cavitation in pipes and drains? (HD7)**

Cavitation refers to the formation of air bubbles in fluid in low-pressure condition which is lower than the saturation pressure. It is a potentially damaging condition in which the fluid in pipes or sewers is at high velocities. By Bernoulli's Equation, at high flow velocities, the pressure head of fluid is reduced accordingly. As the fluid pressure is less than saturation pressure, dissolved gases are released from the fluid and these air bubbles will suddenly collapse when the flow enters into a region of higher pressure. This produces a high dynamic pressure which causes damage to the pipelines due to its high frequency.

**17. Should we cater for manhole loss in design? (HD8)**

Manholes are provided in locations where there are changes in size, direction and gradient of gravity pipelines. In normal practice for straight pipelines manholes have to be installed at a certain spacing to facilitate the maintenance of pipes. With the introduction of manholes, there are various reasons which account for the manhole loss [9]:

(i) The sudden expansion of inflow into manholes and the sudden contraction of flow out of manholes lead to significant energy losses.
(ii) It is not uncommon that several pipes may be connected to the same manhole. As such, the intermixing of flow takes place inside the manhole and this leads to head losses.
(iii) Flow inside the manholes may be designed to change directions which contribute to additional losses.

**18. 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.

**19. What is "residual flooding"? (HD9)**

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".

**20. How to achieve flood prevention by On-site Stormwater Detention? (HD9)**

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.

**21. In scale models, should Froude Number or Reynolds Number be adopted to obtain similarity between model and prototype?**

Froude number is used when gravitation forces is predominant in the channel flow. Reynolds number is adopted when viscous forces are predominant in the channel flow. It is almost impossible to make Froude number and Reynolds number identical in model and prototype. Therefore, the use of these numbers should be judged case by case. For instance, in open channel flow Froude Number is used in the scale models as gravitation forces is predominant.

## 22. 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.

# Level Two (Advanced FAQs)

## ***Part II: Cracking and Tests***

### 1. What are the possible causes of longitudinal and circumferential cracking in concrete pipes? (C1)

Concrete pipes are designed to crack in tensile zone so that steel reinforcement could take up the tensile stress. Flexural stresses are developed at the top and bottom inside surfaces and on the outside surface at the sides. As such, they are the potential locations of longitudinal cracks.

Longitudinal cracks are formed as a result of excessive soil and traffic loads or inadequate pipe bedding. Visible longitudinal cracks observed at top and invert inside the pipe should be more severe than those on the outside because tensile stress occur at top and invert portion of the inside of the pipes. On the contrary, longitudinal cracks formed outside the pipe at the sides of concrete pipe should be more severe than those on the inside.

Multiple longitudinal cracks with small crack width (e.g. 0.15mm) is acceptable which indicate effective transfer of stress from concrete to steel. Care should be taken when discovering a single wide longitudinal crack. Circumferential cracks may occur owing to loads imposed during construction and uneven bedding. It may also be caused by the relative movement of another drainage structure connecting to the concrete pipe. Circumferential cracks do not generally affect the load-carrying capacity of concrete pipes.

### 2. What is the importance of uniform support for precast concrete pipes? (C1)

Concrete pipes are designed to be uniformly supported along the length to carry vertical loads on its top. They are normally not intended to serve as a beam to carry loads in longitudinal direction under poor ground supports (i.e. high and low spots in bedding). Under cantilever beam action and simply support beam action as shown in the diagram below, circumferential cracks would develop in concrete pipes. Circumferential cracking develops in concrete pipes only when pipe bedding becomes non-uniform. Therefore in unstable ground conditions such as soft spot and hard foundations, care should be taken to provide ***firm and even support*** to concrete pipes.

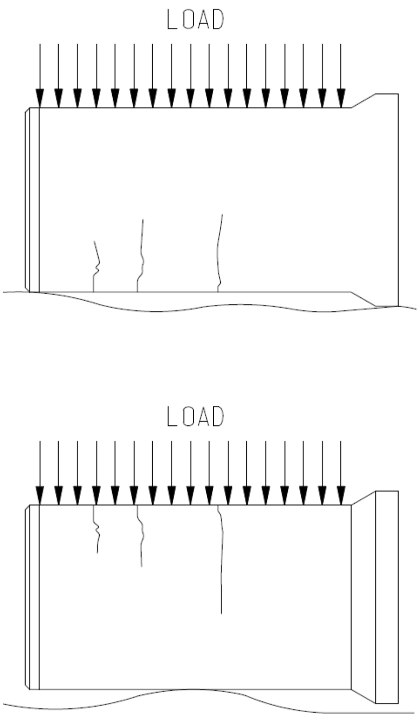



**Fig. 3.1 Crack formation in non-uniform support**

**3. During the time of construction, cracks are likely to develop in small diameter concrete pipes. Why? (C1)**

During the construction of new pavement, vibratory roller and heavy equipment are needed to compact the filling material and bituminous material. These heavy equipment could generate very high impact load with short duration on the concrete pipes. It requires an extremely even and uniform support in order to safely support these loads. Therefore, it is not uncommon that construction vehicles are primary source of crack-producer and it usually results in the formation of circumferential cracks.

**4. Should air test or water tests be selected to test the leakage of constructed gravity pipelines? (T1)**

For gravity pipes, air tests or water tests are carried out after completion of laying and jointing of the pipes. These tests are conducted to check the watertightness of joints and to ensure the pipelines are free from damage where leakage may occur.

Air test has the advantage that the test itself is simple and faster to be carried out. It does not require the disposal of significant quantities of water used in the test which is a mandatory requirement for water test. However, in case leakage exists in the constructed segment of gravity pipelines, the position of leakage can hardly be located in air test. Moreover, the rate of water leakage cannot be determined from air tests. In addition, air test is readily affected by atmospheric condition because air has a relatively high coefficient of thermal expansion. The test is also influenced by the moisture condition of the test pipelines because it affects the passage of air through the pipelines.

For water test, though it is comparatively slow, it can detect the location of water leakage. However, the leakage rate results from water test may not truly reflect its actual leakage because pipeline materials like concrete and clay are porous and would absorb water during the test.

**5. What is the purpose of carrying out water absorption test for precast concrete pipes? (T2)**

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.

**6. Which types of soils are unsuitable for testing under sand replacement test? (T3)**

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.

# 5. Module Four: Marine Works

## Objectives

| Element | Description | Objective No. |
|---|---|---|
| **Piers and Dolphins** | | |
| Piers | Different configurations | P1 |
| | Beams and deck | P2 |
| Dolphins | Flexible dolphin/rigid dolphin | D1 |
| | Breasting dolphin/mooring dolphin | D2 |
| Fenders | Design | F1 |
| **Reclamation** | | |
| Reclamation | Bunds | R1 |
| | Mud waves | R2 |
| | Different approaches | R3 |
| | Dredgers | R4 |
| | Geotextiles | R5 |
| | Settlement | R6 |
| | Vibro-compaction | R7 |
| | Vertical drains | R8 |
| | Slip joints | R9 |
| **Marine Piles** | | |
| Marine Piles | Design | MP1 |
| | Soil Plug | MP2 |
| | Reinforced concrete infill | MP3 |
| | Driving operation | MP4 |
| **Design of Marine Structures** | | |
| Fender Design | Berthing energy absorbed by piers | FD1 |
| | Berthing energy absorbed by water | FD2 |
| | Stiffness | FD3 |
| Armour | Stability | A1 |
| | Hudson's formula/Van der Meer formula | A2 |
| Marine Concrete | Cover | MC1 |
| Piles | Wave force on piles | P1 |

# Level One (Core FAQs)

## Part I: Piers and Dolphins

### 1. Which configuration is better, finger jetty or T-shaped jetty? (P1)

Finger jetty is a more efficient pier structure because it could accommodate vessels at both sides of the jetty. However, there should be sufficient water depth as the berths at finger jetty is relatively close to shoreline when compared with T-shaped jetty so that it is anticipated that vessels are required to berth at shallower water. Moreover, there should be no cross current to enhance berthing at both sides of finger jetty. Also, as mooring points are often located on the jetty, leads are not ideal for larger ships.

T-shaped jetty allows higher water depth for vessels to berth. Moreover, with the installation of breasting dolphins and mooring dolphins, it allows the berthing of larger vessels.


**Fig. Finger jetty and T-shaped jetty**

### 2. Why is shallow bedrock condition unfavorable for open berth piers?

The most severe load on piers generally is the horizontal load due to berthing of large vessels. Since the widths of open berth piers are relatively small so that they provides 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 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, then the little soil cover may result in insufficient lateral resistance to the berthing load.

**3. Why are high and narrow beams not desirable in concrete piers? (P2)**

Based on past experience in other countries (Carl A. Thoresen (1988)), high and narrow beams after several years of construction showed signs of serious deterioration at the bottom of the beams. However, the deterioration of pier slabs was not significant when compared with that of the deep beams. The main reason to account for this is due to the close proximity of the deep beams to the sea level. To avoid these problems, either beamless slab or wide with shallow beams are normally designed.

**4. Shall a layer of wearing course or additional thickness be designed on the surface of piers? (P2)**

In the design of piers, consideration should be given to the effect of wearing action caused by passengers, other traffics and even sometimes vehicles. In maritime environment, the durability and integrity of concrete is detrimental to the servicing life of piers because it acts an essential barrier to chloride attack. However, in view of these gradual wear and tear generated by the loading traffic, some forms of surface protection should be provided on top of pier surface like wearing course or additional increase in concrete cover.

**5. In mooring of vessels, wire ropes or fibre ropes are commonly used for tying the vessels to mooring system. It is not recommended to use them together in mooring. Why?**

Mooring lines are provided by vessel while the shore provides the mooring points.

Wire ropes provide a more rigid mooring system than fibre ropes. When a high degree elastically is required, fibre ropes would be a better choice. The mixed usage of wire ropes and fibre ropes is not recommended because of the uneven tensioning of ropes. Owing to different material properties associated with wire ropes and fibre ropes, it is almost impossible to allocate uniform tension on both types of wires. As a result, there is possible occurrence of overloading in some ropes.

**6. What is the difference in design philosophy between flexible dolphin and rigid dolphin? (D1)**

(A) Rigid dolphin

The impact energy of vessel is absorbed mainly by fender. As such, the dolphin itself is designed as a rigid structure with a group of piles. The piles serve to transfer the reaction force from fender system to the foundation soils. The design of rigid fender is similar to other structures and the strength and stiffness of rigid dolphin should be sufficient to withstand berthing forces without causing excessive deformations.

(B) Flexible dolphin

The impact energy of vessel is absorbed by lateral deflection of piles. The dolphin itself performs both the functions of *fender and berthing structure*. Flexible dolphin is particular suitable in deep water region because the energy absorption capacity is a function of pile length. The pile stiffness could not be designed to be too low because large deflection of pile may occur so that the pile may touch the jetty or the vessel may touch the pile. On the other hand, pile stiffness could not be designed to be too high because of potential yielding of piles or vessel's hull.

**7. What is the difference between breasting dolphin and mooring dolphin? (D2)**

A dolphin is an isolated marine structure for berthing and mooring of vessels. It is not uncommon that the combination of dolphins with piers could drastically reduce the size of piers.

Dolphins are generally divided into two types, namely breasting dolphins and mooring dolphins. Breasting dolphins serves the following purposes:

(i) Assist in berthing of vessels by taking up some berthing loads.
(ii) Keep the vessel from pressing against the pier structure.
(iii) Serve as mooring points to restrict the longitudinal movement of the berthing vessel.

Mooring dolphins, as the name implies, are used for mooring only and for securing the vessels by using ropes. They are also commonly used near pier structures to control the transverse movement of berthing vessels.

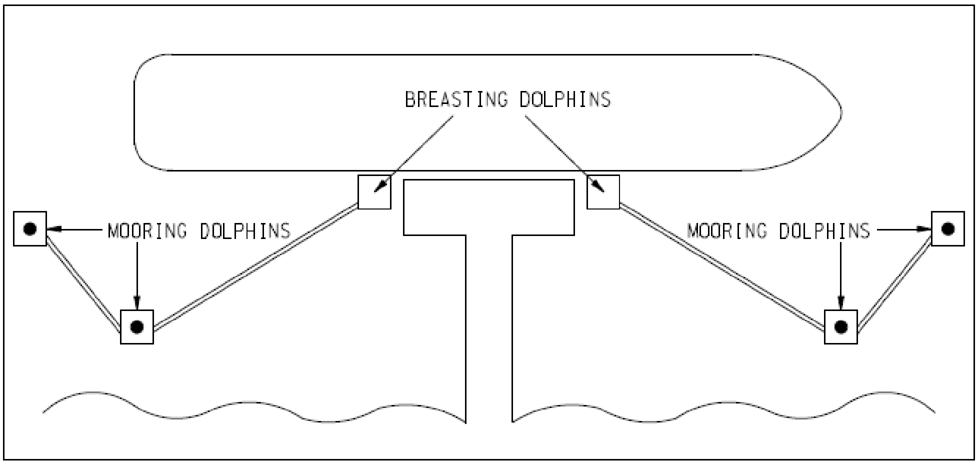



**Fig. Breasting dolphin and mooring dolphin**

## 8. Should dolphins be designed in a rigid manner, i.e. resting on several raking piles?

In designing dolphins, they are normally supported on a system of three to four raking piles. This in essence is a rigid structure and exhibits little flexibility e.g. movement against impact and berthing loads by vessels. In fact, this kind of design may not be desirable in terms of maintenance because the dolphins are readily susceptible to damage by high berthing vessels. To rectify this situation, some energy absorption devices like rubber/plastic fenders have to be installed to reduce the impact load deriving from its own deflection. On the other hand, by designing dolphins as flexible structures capable for allowing slight deflection, it helps to reduce the large forces generated during berthing of vessels. In this connection, one way of designing dolphins as flexible structures is by provision of a single pile only.

Note: For a rigid structure, it takes up external loads without undergoing excessive deformations.

## 9. Why is sulphate-resisting cement not used in marine concrete?

The main components of Portland cement are tricalcium silicate, dicalcium silicate, tricalcium aluminate and tetracalcium aluminoferrite. In sulphate-resisting cement, it contains a low amount of tricalcium aluminate in order to avoid sulphate attack. Otherwise, tricalcium aluminate would react with sulphates to form calcium sulphoaluminate and gypsum that cause expansion and crack the concrete structure.

However, for marine concrete sulphate-resisting cement should not be used because tricalcium aluminate has high affinity for chloride ions. This is based on the possible reaction of chloride ions and tricalcium aluminate to form calcium chloroaluminate hydrate as suggested by P. Kumar Mehta (1991) and the reduction of which may increase the rate of chloride attack to the concrete marine structure and result in faster corrosion of steel reinforcement in marine structures.

**10. What is heeling during vessel berthing?**

When a vessel berths on a fender system at a pier, the point of contact of the berthing ship may be above or below the centre of gravity of the ship. During the berthing operation, some kinetic energy is dissipated in work done to heel the ship i.e. the work done to bring the ship an angle of heel. This energy is normally a small fraction of total berthing energy and therefore it is normally not considered in design. However, designers should pay attention to the possible hitting of the berthing structure by the vessels in case the contact point is well above water level [19].

**11. What is the difference in application of surface-protecting fenders and energy-absorbing fenders? (F1)**

Surface-protecting fenders are fenders that induce high reaction forces to berthing structures for the energy absorbed while energy-absorbing fenders are fenders which transmit low impact to berthing structures for the energy absorbed (Carl A. Thoresen (1988)). In fact, the principal function of fenders is to absorb the berthing energy and transmit a force to the structures without damaging them. Therefore, in open berth structures, it is desirable to use energy-absorbing fenders to reduce the loads acting on the relatively flexible structures. On the other hand, for solid berth structures the usage of surface-protecting fenders is adequate because they are capable of taking up large berthing loads.

**12. What is the difference between weight chain, shear chain and tension chain in fender system? (F1)**

The weight chain is used to sustain the weight of face and frontal panel. Shear chain help protect the fender from damage while the fender is in shear deformation and they are orientated at 20 - 30° to the horizontal.

Tension chain serves to guard the fender against damage when the fender is under compression.

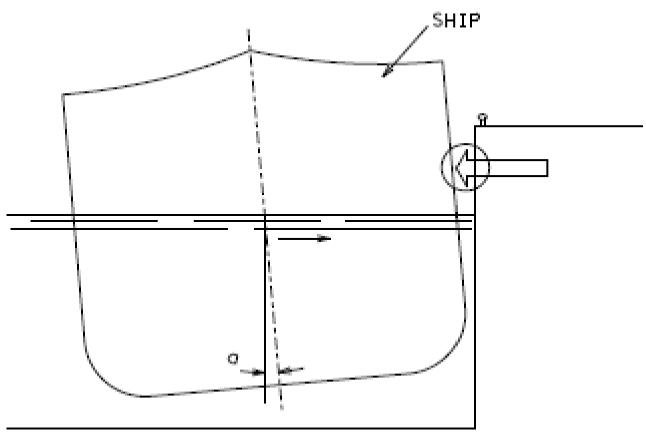



**Fig. Heeling of a vessel**

**13. In connecting fenders to pier structures, should single lock nuts or double lock nuts be used? (F1)**

In many pier structures the connection of fenders to piers is achieved by using single lock nuts. However, they do not perform well because some timber fenders loosen more easily when subject to vibrating loads due to berthing, wave and tidal actions. To solve this problem, double lock nuts should be adopted as they prove to function satisfactory in other structural elements which are subject to frequent vibration loads.

Note: Double lock nuts mean two nuts are adopted in a single bolt connection between fenders and marine structures.

**14. What are the pros and cons of using timber fenders, plastic fenders and rubber fenders? (F1)**

Timber fenders:
They are low in strength and are subject to rotting and marine borer attack. Moreover, they have low energy absorption capacity and the berthing reaction depends on the point of contact. The contact pressure between fender and vessels are high. They are considered to be environmentally unfriendly because they consume tropical hardwoods in their production.

Plastic fenders:

Their strength is similar to that of timber fenders but they have relatively high abrasive resistance. They are resistant to chemical and biological attack. Their energy absorption capacities are moderate and the berthing reactions are also dependent on the point of contact. The reaction is lower when compared with timber fenders for a given energy absorption. They are considered to be environmental friendly because they are manufactured from recycled material.

Rubber fenders:
They possess high abrasive resistance and are also resistant to most biological and chemical attacks. They have moderate to high energy absorption capacity and the energy absorption performance is independent of the point of contact. Similar to plastic fenders, they are also environmental friendly products.

## Level One (Core FAQs)

### *Part II: Reclamation*

**1. What are different approaches for reclamation in deep water region and shallow water region? (R3)**

To illustrate the different approaches adopted for reclamation in deep water and shallow water region, the following example is used:

In deepwater region, consider the seabed level is –8.5mPD. After laying of geotextiles and 1.5m thick sand blanket, the top level of sand blanket is about –7mPD. Split barges are deployed for dumping public fill to –2.5mPD. Afterwards, end dipping of public fill by trucks will be carried out up to +2.5mPD which is the designed reclamation level. Between level –2.5mPD and +2.5mPD, it is too shallow for split barges to enter the water, thus the method of end dipping is used instead.

For shallow water region, the seabed level is taken as –5.5mPD in this example. With the laying of geotextiles and 1.5m sand blanket into position, the top level of sand blanket is about –4mPD. In this case, split barges are also used for reclamation work between the level –4mPD and –2.5mPD. After that, if end dipping is used for reclamation work above –2.5mPD, then in considering the relative thin layer of fill above seabed (1.5m sand blanket + 1.5m sand blanket), it stands a high chance that mud wave would occur in seabed. Therefore, half-loaded derrick barges are employed for reclamation up to level 0mPD. With a thicker layer of public fill now, end dipping can then be used for reclamation between 0mPD and +2.5mPD.

This above reclamation sequence is just an example to demonstrate the different considerations for reclamation in deep water and shallow water region.

**2. In dredged reclamation, what are the considerations in selecting between trailer suction hopper dredgers and grab dredgers? (R4)**

Trailer suction hopper dredgers are vessels which remove material off the seabed through hydraulic suction by using pumps. During the dredging operation, a mixture of soil and water is transported through suction pipe to storage hoppers. Significant turbulence inside the hoppers keeps the dredged mixture in suspension and this should be minimized to enhance the material to settle swiftly prior to the process of overflowing. Trailer suction hopper dredgers are mounted with draghead or dragarm pumps which increases the dredging depth and trims down the occurrence of cavitation as suggested by John B. Herbioh (1992). This machine is limited to dredging relatively low-strength material. Moreover, they cannot be deployed in very shallow waters and instead grab dredgers should be used. However, its dredging capacity is higher than that of grab dredger and it can be mobilized in relatively deep-water region.

Trailer suction hopper dredgers are renowned for their mobility, versatility and capability to operate in unfavorable sea conditions.

**3. What is the purpose of formation of bunds in reclamation? (R1)**

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.

**4. In case mud waves occur during reclamation, what are the possible solutions to rectify the situation? (R2)**

(i) Option 1 – Complete Removal of All Disturbed Mud
To remove all disturbed mud once mud waves occur is the fastest way to treat the problem. After that, filling material is used for replacing the disturbed mud. However, this option is a rather expensive option because it involves dredging of all disturbed mud and replacement of large amount of fill.

(ii) Option 2 – Accelerated consolidation of Disturbed Mud
This option involves placement of surcharging loads on top of mud waves, together with installation of band drains to accelerate the consolidation of disturbed mud. This method suffers from the drawback that sufficient long time is required for the consolidation process of mud.

(iii) Option 3 – Partial Removal of Disturbed Mud
This option is a combination of the first two options in which the top weak layer of mud is removed while the lower mud is treated with surcharging with band drain installation.

Note: Mud waves refer to excessive displacement of mud due to successive slip failure during reclamation.

**5. What is the importance of geotextiles and sand in reclamation works? (R5)**

For geotextiles used in reclamation, they serve mainly the following two purposes:
(i) they separate reclamation fill from marine mud;
(ii) they may act as reinforcement to enhance the stability of reclamation. However, the reinforcement function is still under heated debate because its performance as reinforcement depends on several factors like the directional strength of woven geotextiles and damage effect by installation of vertical band drains.

For sand:
(i) it spreads the load of future public dump on top of it;
(ii) it acts as drainage path for dissipation of excess pore water pressure for band drain installation.

**6. In reclamation by filling sand, what is the effect of filling operations below mean sea level?**

Filling below mean sea level usually has a low density. The settling sand in standing water would form a loose skeleton leading to a low density. However, as the sand level is rising, the increased load causes reallocation of sand grains in lower layers. As such, after dissipation of excess pore water pressure, it results in increased density.

**7. 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.

**8. Why are observed settlements in reclamation normally larger than calculated? (R6)**

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.

**9. Should silt curtain be designed to touch seabed?**

Silt curtains are impermeable vertical barriers extending from the seawater surface to their designed depths. The curtains are held in a vertical position by the carrier float on their top and a curtain weight at their bottom. A tension cable is designed at the carrier float to resist stresses incurred by currents. Moreover, the silt curtains are anchored to the seabed to hold them in the designed configuration.

In essence, the depth of silt curtains should not be so long and touch the seabed because the bottom segment of the silt curtains would be trapped inside the newly accumulated sediment, thus resulting in the sinking of the curtain. It is difficult to remove these sunken curtains. Moreover, reversal tidal and current actions may cause movement of bottom region of curtains which stir up the settled suspensions and create additional turbidity.

**10. Vibrocompaction is carried out to loose sand after reclamation by filling sand. How to determine the in-situ density of filled sand? (R7)**

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.

**11. Why do compliance test be carried out some time after the completion of vibrocompaction?(R7)**

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.

**12. 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.

**13. 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.

**14. What are the potential problems of smear around vertical drains? (R8)**

Smear zones are generated during the installation of vertical drains in which the zone of soils surrounding the band drains are disturbed. Soils in the smear zones are remoulded during the installation process and the effectiveness of band drains is reduced. For instance, the compressibility of surrounding soils is increased and this brings about the reduction of their permeability. In essence, with the reduced permeability of soils around band drains, it takes longer time to complete the consolidation process.

To prevent the formation of smear zones, the raising and lowering of mandrel during drain installation should be minimized. Moreover, soil disturbance can be controlled by avoiding the use of vibratory hammers which serve to drive the drains into the ground [35].

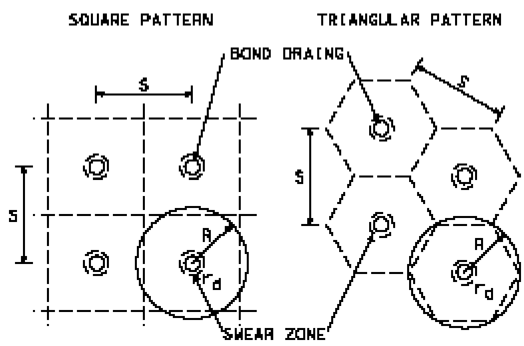



**Fig. Smear zones in band drain**

**15. For drained reclamation, what is the significance of smear zone induced by installation of band drains? (R8)**

During installation of band drains, smear zones are created in which a zone of soil surrounding the band drains are disturbed. The compressibility of surrounding soils is increased and it results in the reduction of their permeability. In fact, the surrounding soils are remoulded during the installation process and the effectiveness of band drains is reduced. In essence, for the reduced permeability of soils around band drains, it takes longer time to complete the consolidation process.

**15. What are the considerations in selecting marine plants and land plants for installation of band drains? (R8)**

For installation of band drains by marine plants, it must have sufficient water depth to accommodate the marine plants in the first place. However, due to the effect of tides and waves, the establishment of the position for installation of band drains and the subsequent installation works may be affected. In addition, the establishment cost of marine plants is higher than that of land plants.

For installation of band drains by land plants, difficulty may be encountered during the installation of band drains through the reclaimed layer e.g. C&D material. Land plants may take longer construction time due to the above-mentioned difficulty. Sometimes when the supply of public fill is increased suddenly, it may be preferable to place these fill immediately into position and in this situation the installation of band drains (originally installed by marine plants) is delayed so that the construction of band drains is changed to using land plants.

**17. What is the purpose of providing shoes in prefabricated drains? (R8)**

Shoes are normally installed in prefabricated drains for the following reasons [35]:

(i) It avoids the entry of soils into the mandrel by sealing it during the installation of drains.
(ii) It performs like an anchor to retain the drains at the designed depth and to stop the drains from being pulled out during the withdrawn of mandrels after driving the mandrels into ground.

However, the inclusion of shoes in prefabricated drains tends to aggravate the problem of smear effect because the shoes are usually larger in size than mandrels.

**18. Vertical drains can be installed in square and triangular patterns. Which pattern is better? (R8)**

Vertical drains are commonly installed in square and triangular patterns. The zone of influence of vertical drains (R) is a function of drain spacing (S).

For drains in square pattern: R = 0.546S
For drains in triangular pattern: R = 0.525S

The advantages of square pattern are that it is more convenient to lay out and manage on site. However, triangular pattern is the most popular one because it provides a more uniform consolidation between drains than square pattern.

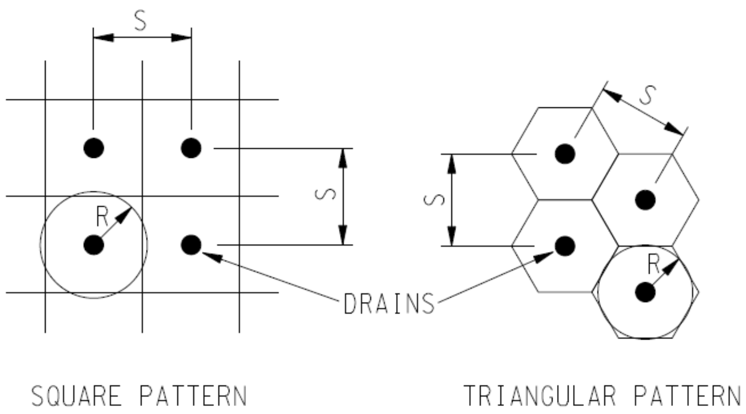



**Fig. Square pattern and triangular pattern of vertical drains**

**19. How does overflowing in trailing suction hopper dredger affect the water regime? (R4)**

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

**20. 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.

**21. Geotechnical Instrumentation is frequently employed for monitoring the condition of reclamation. Sometimes two piezometers are installed inside the same borehole. What is the reason behind this?**

For standpipes, they normally contain one plastic tube between its intention is to measure water level only. However, for piezometers, they are used for measuring pore water pressure in a certain depth below ground. For instance, if there are two clayey layers below ground at different depths, a multiple piezometer including two separate piezometers may be sunk at the same borehole to determine the pore water pressure at these layers respectively. This arrangement has the advantage that it saves the cost of installation of separate boreholes for several piezometers. However, the installation of multiple piezometers within the same borehole is affected by occurrence of leakage along the pipes as suggested by Marius Tremblay (1989).

**22. In case a road passes through a reclaimed land and an existing land, what is the main concern regarding the design of pavements?**

For an existing land, it is anticipated that there will be no major settlement within the design life of pavement structures. However, for a recently reclaimed land, even with surcharging and installation of vertical drains, some settlement will still occur after the construction. If a road pavement has to be constructed connecting these two areas, special design has to be made in this transition region. In order to avoid the occurrence of differential settlement which may damage the pavement structure, a transition slab may be designed to accommodate such movement (J. S. M. Kwong (1996)).

**23. What are the functions of slip joints in blockwork seawalls? (R9)**

Slip joints are joints which are formed through a complete vertical plane from the cope level to the toe level of seawalls. These joints are designed in blockwork seawalls to cater for possible differential settlements between adjacent panels of seawalls. The aggregates inside the half-round channels in slip joints allow for the vertical movements induced by differential settlement and at the same time providing aggregate interlocking forces among adjacent panels of seawalls to link the panels in one unit against the lateral earth pressure exerted on seawall.

Besides, slip joints provide a path for the relief of water pressure developed and allow the lateral movement (e.g. contraction) due to seasonal variations.

Note: For details of slip joints, reference is made to CEDD Standard Drawing No. C3008C.

# Level Two (Advanced FAQs)

## ***Part I: Marine Piles***

### 1. Why are most marine piles circular in cross section? (MP1)

For marine piles, there are several options available for selection, namely H-piles, circular pipes and box piles.

However, only circular piles and box piles are suitable for marine application because of the following two reasons suggested by G. M. Cornfield (1968):

(i) Circular piles and box piles possess high column buckling strength. For marine structures like jetties, piles are well above seabed level and therefore the column buckling effect is significant when compared with other structures. Therefore, it is essential to use pile sections which have relatively high buckling strength in piers.
(ii) Circular piles and box piles display high energy absorbing capability. For marine structures like dolphins and fenders, which require substantial amount of berthing energy to be absorbed, these piles sections are inevitably good choices.

In marine structures, it appears that circular sections prevail over the box sections. The main reason is that the range of section available for selection of circular piles is more than that of box piles.

### 2. For marine pile type of steel tubular piles with reinforced concrete infill, minimum toe level is often specified in contract drawings. What is its purpose? (MP1)

The purpose of minimum toe level is two-fold:

(i) In detailed design stage, ground investigation should be 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.
(ii) It provides sufficient length of soils for lateral and uplift resistance.

Note: Minimum toe level refers to the minimum level that a marine driven pile should be driven into seabed.

**3. What is the problem in traditional marine piling system of steel tubular pile with concrete infill and what are the possible remedial measures? (MP1)**

In the design of marine piles of steel tubular piles with concrete infill, loads from pier deck are taken up by steel tubular piles before the occurrence of corrosion of steel piles above seabed. In fact, it is assumed that steel piles above seabed level will all be corroded after a certain year. The load transfer mechanism after complete corrosion of steel pile above seabed is as follows: loads from pier deck are taken up by concrete infill above the seabed level. Below the seabed level, loads would be transferred to steel piles through frictional forces between concrete infill and steel casings.

However, substantial radial shrinkage and contraction occurs after concreting of concrete infill and this will hinder the load transfer from the concrete infill to steel piles because the bond may be ruptured by radial shrinkage. It is in doubt if frictional forces can be properly developed in this situation. To solve this problem, shear keys could be installed at regular spacing inside steel piles to ensure their rigid connection with concrete infill. Alternatively, expanding agents may be adopted in concrete mixes to ensure that there is no shrinkage after the concreting process.

**4. Why are steel tubular marine piles often driven open-ended? (MP4)**

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 depth. 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.

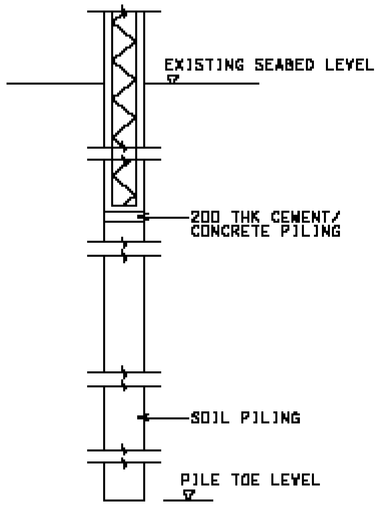



**Fig. Typical details of marine piles**

## 5. What is the mechanism of formation of soil plugs in marine driven steel piles with reinforced concrete infill? (MP2)

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 frictional forces exceeding the bearing capacity of soils at pile toes. Consequently, the soil plug formed is brought down by the piles. Reference is made to M. J. Tomlinson (1977).

Note: A soil plug is a column of soil formed at the bottom portion of marine pile type of steel tubular piles with reinforced concrete infill.

## 6. How does soil plug in open-ended tubular piles affect its loading carrying capacity? (MP2)

In marine piles, the tubular piles are sometimes purposely designed to be open-ended to facilitate deeper penetration. In this mode of pile formation, soil plug is formed inside the piles.

The plugging ratio (the ratio of length of soil plug to the length of pile penetration) affects the load carrying capacity of piles. It was demonstrated by experiments that the end bearing capacity decreases with an increase in the plugging ratio. Moreover, close-ended piles display a higher end bearing capacity than open-ended piles as close-ended piles prevent soil from entering the piles and force them around the pile tip leading to a higher stress state in this region.

**7. Why are sleeves often installed inside the lower part of open-ended driven piles? (MP1)**

In marine piles, open-ended piles are more often used than close-ended piles to enhance longer length of piles installed. This is essential to provide better lateral resistance against berthing loads and other lateral loads in marine structures.

To enhance longer length of driven piles installed, sleeves can be employed inside the lower part of open-ended driven piles. The sleeves lead to improved drivability with more soil entering the pile to from soil plug. The longer is the sleeves, the higher is the plugged length. This is because there is higher stress release experienced by soils flowing past the sleeves.

**8. What is the significance of reinforced concrete infill in marine piling system of steel tubular pile with reinforced concrete infill? (MP3)**

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.

**9. Can soil plug be removed in marine piling system of steel tubular pile with reinforced concrete infill?**

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 frictional 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.

**10. Why “inadequate pile founding level” commonly occurs in piles of piers? (MP1)**

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.

**11. For typical pile bents in marine piers, how is vertical loads related to horizontal capacity of the pile bents? (MP1)**

Let’s consider a pile bent with a top slab supported by two ranking piles, each inclining at an equal angle to the pier slab. In designing such a system, truss action is normally adopted to analyze the pile bent. When the reaction forces of these piles, horizontal forces (e.g. due to berthing and deberthing of vessels) and vertical forces (e.g. superimposed deck loads) are analyzed by drawing a force polygon, it is noted that lateral resistance of the pile bent is dependent on the vertical load, i.e. lateral resistance is small when vertical loads are high.

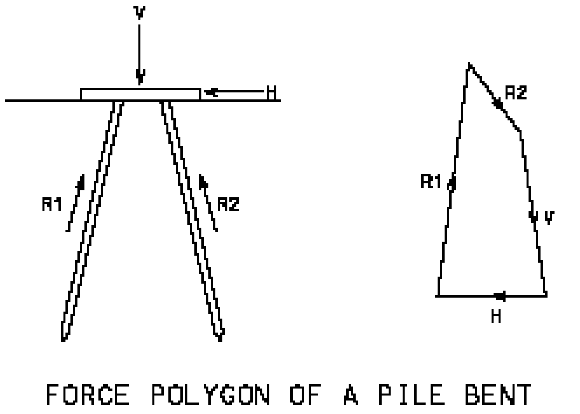



**Fig. Force polygon of pile bent**

**12. For underwater concreting, tremie pipes are normally used with the aid of hoppers. Sometimes tubes are inserted inside the hoppers. Why?**

In placing concrete by tremie pipes, hoppers are connected to their top for receiving freshly placed concrete. However, air may be trapped inside the tremie pipes if there is rapid feeding of fresh concrete. To release the trapped air inside the tremie pipes, hoses (called ventilation tubes) are inserted and lowered down through the hoppers. Reference is made to Carl A. Thoresen (1988).

## Level Two (Advanced FAQs)

### ***Part II: Design of Marine Structures***

**1. In fender design, when calculating the berthing energy absorbed by fenders, should engineers take into account energy absorbed by piers? (FD1)**

The design of a fender system is based on the principle of conservation of energy. The amount of energy brought about by berthing vessels into the system must be determined, and then the fender system is devised to absorb the energy within the force and stress limitations of the ship's hull, the fender, and the pier.
Firstly, the energy released by the largest/heaviest vessel allowed to use on the pier is determined to be delivered to the pier by first impact. Then, the energy that can be absorbed by the pier would be calculated. For pier structures that are linearly elastic, the energy is one-half the maximum static load times the amount of deflection. However, in case the structure is extremely rigid, it can be assumed to absorb no energy.

*The energy to be absorbed by fender system should be the total energy of berthing vessels deducting the energy absorption by pier structures.* Finally, a fender system capable of absorbing the amount of energy without exceeding the maximum allowable force in the pier should be chosen from fender product catalogue.

**2. Why do vessel operators choose to contact the fender system at its bow instead of mid-ship location during berthing operation?**

When calculating berthing energy of vessels, there is a factor called “eccentricity factor” which accounts for different berthing energy when the vessel contact the fender system at different locations of the vessel.

For instance, for mid-point berthing the eccentricity factor is unity which means there is no loss of berthing energy. For third-point berthing and quarter-point berthing, the eccentricity factor is 0.7 and 0.5 respectively. In fact, engineers always attempt to reduce the amount of berthing energy to be absorbed by fender system and pier structures. As such, it is recommended for vessels to contact fender system at its bow or stern because the reaction force would produce a rotational moment to the vessel which dissipates part of vessel’s energy.

**3. Can water help dissipate part of berthing energy? (FD2)**

Depending on the configuration of pier, water could help dissipate part of berthing energy. For instance, for closed docks in which there is a solid wall going down directly to the bottom of seabed, the quay wall will push back all the water that is being moved by the vessel and creates a cushion effect which dissipates part of berthing energy (10-20%). On the contrary, for open dock in with piles beneath and water can flow through the underside of piers, there shall be no cushion effect of water.

Similarly, the larger is the draft of vessels, the less trapped water can escape under the vessel so that the cushion effect of water can be enhanced to dissipate part of berthing energy.

**4. Should stiff or soft fenders be designed for berthing in piers? (FD3)**

The elasticity of fenders is related to the ability to release the stored energy during berthing of vessels. However, it has no effect on the reaction force and the deflection of fender system. The amount of energy that a fender can absorb is dependent on the reaction-deflection curve and is represented by the area under the curve. The higher is the reaction force, the higher amount of energy would be absorbed by the fender provided that the resistance of ships' hull is sufficient to withstand the force without permanent deformations. Although stiff and soft fender may have the same deflection under the same maximum reaction force acting on the berthing vessel, the amount of energy absorbed by stiff fenders is much higher than that of soft fenders. Consequently, stiff fenders should be employed for berthing purpose.

On the other hand, in mooring operations where vessels are constantly subject to wave action, it is desirable to keep the tension force on the rope to a low value. In this connection, it is recommended to use soft fenders.

**5. Should small vessels be considered in the design of fenders?**

Smallest vessel should also be taken into consideration when designing fender system. In vertical orientation of fender system, the types and sizes of all vessels should be considered taking into account the tidal effect at that region. To conduct proper design of fender system, engineers must consider the height and draft of both the smallest and largest vessels to determine the point of contact on the fender. It is not uncommon that design of fender system considers only the largest vessels berthing in the pier which should be avoided as it might not function for smaller vessels berthing in dock.

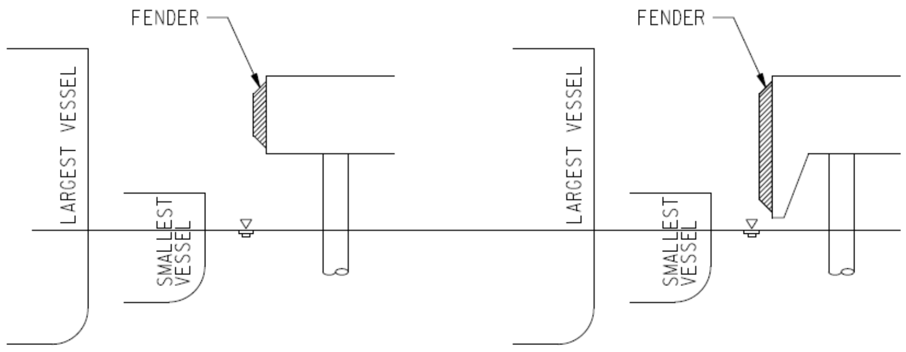



**Fig. Effect of smallest vessels in fender design**

### 6. Why is it desirable to select fenders with low reaction force? (FD1)

It is always in the interest of engineers to select fenders with high energy absorption with low reaction force. The reaction force is an important factor in the design of quay walls because sufficient savings could be resulted from low reaction force exerted by fenders. In many fender systems such as rubber fender, load-spreading panel is often adopted to cater for high reaction force. Hence, the use of fenders with low reaction force gets rid of the need of load-spreading panel so that significant economies could be made.

### 7. What are the factors determining the stability of a single armour unit? (A1)

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.

**8. Hudson's formula and Van der Meer formula are commonly used in the design of armour. Which one is a better choice? (A2)**

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).

**9. Should the Morison equation or diffraction analysis be adopted in determining wave force on piles? (P1)**

The choice between the Morison's equation and diffraction analysis in determining the wave forces on piles depends on the ratio between the diameters of piles to wavelength. If the ratio between the diameter of piles to wavelength is less than 0.2, the Morison equation is usually recommended. The reason behind this is that the effect of viscosity and separation is significant below this ratio. On the contrary, if the ratio between the diameter of piles to wavelength exceeds 0.2, the waves are scattered with negligible occurrence of separation. As such, diffraction analysis is adopted to calculate the wave forces on piles.

**10. Why is larger concrete cover (e.g. 75mm) normally used in marine concrete? (MC1)**

In marine environment, the cover to reinforced concrete in exposure zones is specified to be much larger than normal concrete (e.g. 75mm). Based on past experimental studies, the concrete cover is identified to be directly related to the corrosion failure of marine structures.

In Eurocode 1 under Ultimate Limit State the probability of failure is less than $10^{-4}$ while under Serviceability Limit State the probability of failure is less than $10^{-2}$. For corrosion aspects, it is normally classified under SLS state. For OPC concrete with 50mm cover, the corrosion probability is found to be more than $10^{-2}$. However, with the use of 75mm cover, the corrosion probability is about $10^{-3}$, which fulfills the Eurocode requirement.

**11. What is the design level of landings in piers?**

Landings are designed as resting place for passengers during berthing and deberthing of vessels. In general, landings are provided near mean high and mean low water levels to facilitate embarking and disembarking of passengers (BS6349: Part 2: 1988). Therefore, the level of landing steps should be different from place to place because of different mean high and mean low water levels in different locations.

**12. How does the direction of approaching velocities of ships affect berthing?**

One of the major effects of angle of approaching velocities of ships is its influence of the energy to be absorbed by the fender system. Consider several ships berth on the same pier at the same speed but with different angle of approach, though their kinetic energies are the same, the amount of energy absorbed by fender differs. The amount of energy absorbed by fender is [19]:

$$W = \frac{0.5mv^2(k^2 + r^2\cos^2\Phi)}{(k^2 + r^2)}$$

where W= energy absorbed by the fender
m= mass of the ship
v=velocity of the ship
k= radius of gyration of the ship
r= distance of centre of gravity of the ship to the point of contact of the fender
Φ=direction of velocity

Hence, when the direction of approaching velocity of a ship is normal to the fender system (i.e. Φ=90°), the amount of energy absorbed is smaller when compared with that of a ship whose velocity is tangential to the shoreline.

**13. What is immersed tube method for underwater crossings?**

The immersed tube method for underwater crossing involves the following basic construction steps:

(i) Prefabricating long tunnel units (steel shell or concrete) in a dry-dock or shipyard
(ii) Floating and towing the units with removable bulkhead to the site
(iii) Immerse the units in a pre-dredged trench
(iv) Connect the units one by one
(v) Covering the completed tunnel with backfill

Steel immersed tunnel is sometimes adopted because of the ease of fabrication and its relative lightness. Moreover, shorter construction time is required when compared with concrete immersed tubes.

# 6. Module Five: Foundation

## Objectives

| Element | Description | Objective No. |
|---|---|---|
| **Bored Piles and Mini-piles** | | |
| Bored Piles | Cement Plug | BP1 |
| | Water in piles | BP2 |
| | Direct circulation drilling/reverse circulation drilling | BP3 |
| | Bell-out | BP4 |
| | Toe defects | BP5 |
| | Tremie pipe | BP6 |
| | Airlifting | BP7 |
| Mini-piles | Spacing | MP1 |
| | Strain compatibility | MP2 |
| | Grout | MP3 |
| | Left-in casing | MP4 |
| | Post-grouting | MP5 |
| Rock Socket | Load distribution | RS1 |
| **Driven Piles** | | |
| Driven Piles | Hiley's formula | DP1 |
| | Cap block, drive cap and pile cushion | DP2 |
| | Hammers | DP3 |
| | Set | DP4 |
| | $45^o$ load spread rule | DP5 |
| | Followers | DP6 |
| | Hammer efficiency and coefficient of restitution | DP7 |
| | Coating with bitumen | DP8 |
| | Driving sequence | DP9 |
| | False set | DP10 |
| **Pile Tests** | | |
| Pile Load Test | Purpose | PLT1 |
| | Meanings of the mathematical terms | PLT2 |
| PDA Test | Case Method and CAPWAP Analysis | PDA1 |
| | Purpose | PDA2 |
| **Design of Foundation** | | |
| Foundation | Bearing capacity | FD1 |

| **Element** | **Description** | **Objective No.** |
|---|---|---|
| Design | | |
| | Pad foundation, strip foundation and raft foundation | FD2 |
| | Rock anchors and rock sockets | FD3 |
| | Capping beams and ground beams | FD4 |
| | Maximum spacing of piles | FD5 |
| | Point of virtual fixity and critical length | FD6 |
| | Modulus of subgrade reaction | FD7 |
| | Flexible pile cap/rigid pile cap | FD8 |
| | Pinned bases and fixed bases | FD9 |
| | Load distribution in piles | FD10 |
| | Reinforcement in a typical pile cap | FD11 |
| | Negative skin friction | FD12 |
| | Compaction grouting and fracture grouting | FD13 |
| **Miscellaneous** | | |
| Diaphragm Wall | Concreting | DW1 |
| | Bentonite slurry | DW2 |
| Sheet Piling | Tiebacks | SP1 |
| Grout | Stability | G1 |
| | Shaft grouting | G2 |
| Construction Machinery | Reversed Circulation Drills | CM1 |
| | Down the hole hammer | CM2 |
| Deep Excavation | Ground water table | DE1 |
| Prestressed concrete piles | Problems | PCP1 |
| Pile Construction | Compaction | PC1 |
| | Free-falling concrete placement method | PC2 |

# Level One (Core FAQs)

## ***Part I: Bored Piles and Mini-piles***

### 1. What is the function of introducing cement plug before pouring concrete in bored piles? (BP1)

It is a common practice in the construction of bored piles by adding a cement plug before concreting of bored piles takes place. One of the possible explanations is that the cement grout serves as a barrier and protects fresh concrete from being washed away by water inside pile bore. The cement grout helps to set aside water when the first batch of concrete is poured down from the tremie pipe. As such, the quality of fresh concrete is anticipated not to be seriously affected by water and this helps to improve toe imperfection problem.

### 2. Why are bored piles usually cast higher than the required final level?

It appears to be a common construction practice that bored piles are normally cast up to a piling platform much higher than the formation level of pile caps. Then, the excessive pile length has to be cropped and removed to the correct level for the subsequent construction of pile cap. The reasons of such practice are as follows:

(i) It is technically sound to employ such practice because laitance, impurities and poorly-compacted concrete should have migrated to the top of the piles. Therefore, additional length at the top of piles is constructed to accommodate these sub-standard concrete, which are subsequently removed and cropped to leave high quality concrete over the entire length of piles.
(ii) In actual site practice, the details of construction sequence and access arrangements have potential impact on the possibility of constructing piles to correct level without the need of cropping.

### 3. What is the reason of adding water to bored piles? (BP2)

In water bearing ground, some water head (about 1 m) above the existing ground water table is maintained to stabilize the bore during excavation below casings by pumping water to the pile bore. This balanced head condition is created to minimize the possible drawdown of surrounding water table which may affect the stability of nearby structures. Moreover, this helps to limit the possible inflow of water by piping from the base of pile bore.

**4. What is the purpose of maintaining excess water head during excavation for bored piles? (BP2)**

Excavation of bored piles is usually implemented by a hammer grab. The steel casing will be extended by welding or bolting on additional casing and is installed by hydraulic oscillator. Water is pumped into the casing during excavation and excess water head is required to be maintained to prevent any ingress of material at the bottom of casing. Moreover, for excavation below pile casing, the provision of excess water head is essential to maintain bore stability.

**5. What is the difference between direct circulation drilling and reverse circulation drilling? (BP3)**

For direct circulation drilling and reverse circulation drilling, the major difference in drilling method is related to the direction of movement of drilling fluid. For direct circulation drilling, the drilling fluid is circulated from the drill stem and then flows up the annulus between the outside of the drill stem and borehole wall. The drilling fluid that carries the drill cuttings flows to the surface and the subsequent settlement pits. Pumps are employed to lift the cuttings free fluid back to the drill stem.

For reverse circulation drilling, the direction of flow of drilling flow is opposite to that of direct circulation drilling. Drilling fluid flows from the annulus between the drill stem and hole wall to the drill stem. The drilling fluid is pumped to an nearby sump pits where cuttings are dropped and settled.

**6. What are the potential problematic areas in bell-out of bored piles? (BP4)**

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.

**7. There is a general trend that toe defects in bored pile construction have become more serious. Why? (BP5)**

Direct coring method is commonly employed to check the quality and workmanship of bored piles. It is further divided into two main types, namely interface coring (pile/rock interface) and full coring (entire concrete pile length). In the past, the implementation of full-depth coring usually takes place near the pile centre to prevent physical conflicts with reinforcement cage. When placing concrete by tremie method, concrete pouring by tremie pipe is carried out near pile centre and soft and weak materials are usually displaced to the sides of bored piles. As a result, the pile/rock interface at pile centre should indicate the best results. However, with the recent trend of using interface coring through reservation tube attached to the reinforcement cage, there is a high chance that interface coring gives poor results as the reinforcement cage may trap soft and weak materials and they are located far from the pile centre.

**8. Is it totally unacceptable that soil sediments are found at pile toe of bored piles? (BP5)**

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.

**9. What is the purpose of keeping tremie pipe's tip immersed in freshly-placed concrete for about 1m in underwater concreting? (BP6)**

The size of tremie pipe is about 300mm with sections having flange couplings fitted with gasket to prevent water leakage. The tremie pipe should be closed initially to prevent water from entering the pipe. It should be designed with sufficient thickness and weight so that it would not be buoyant when empty inside water.

The placement of tremie concrete is commenced by putting the closed pipe underwater to the location for concreting, followed by partial filling of tremie pipe with concrete. In order to have tremie concrete flowed out of the pipe, it is necessary to fill the pipe with concrete of sufficient height to overcome the water pressure and frictional head. After that the tremie pipe is raised about 150mm to allow concrete to flow out. To enhance sufficient bonding, each succeeding layer of concrete should be placed before the preceding layer has reached the initial set. The tremie pipe should be kept full of concrete up to the bottom of hopper.

The tip of tremie pipe should always be immersed in freshly-placed concrete for at least 1m *to prevent inflow of water into the tremie pipe and to avoid contact of freshly placed concrete with water*. This serves as the seal against water entry. The loss of seal may result in increased flow rate with fresh concrete affected by seawater. The distance that tremie concrete could be allowed to flow without excessive segregation is about 6-20m.

**10. Why shouldn't tremie pipe be left in a position too long without lifting up? (BP6)**

When concrete starts to flow out of tremie pipe, the lifting of tremie pipe should be carried out slowly to avoid disturbance of material surrounding the end of tremie pipe. The mouth of pipe is embedded at least 1m below the concrete surface to maintain the seal.

If a tremie pipe is left in a position too long without lifting up, it would impair the quality of already-placed concrete. The fresh concrete may be placed under the portion of concrete which have set already so that it would raise the mass of already-placed concrete and induce cracks on it.

**11. In piling works, normally founding levels of bored piles are defined by using total core recovery or rock quality designation (RQD). Are there any problems with such specification?**

The use of total core recovery to determine the founding level may not be able to indicate the quality of rock foundation for piles because it depends on the drilling technique and drilling equipment (GEO (1996)). For instance, if standard core barrels are used to collect samples, it may indicate sufficient core recovery for samples full of rock joints and weathered rock. On the other hand, if triple tube barrels are used for obtaining soil samples, samples with joints and weathered rock can also achieve the requirements of total core recovery.

In case RQD is adopted for determining founding levels, it may also result in incorrect results. For instance RQD does not indicate the joints and infilling materials. Moreover, as it only measures rock segments exceeding 100mm, rock segments exceeding 100mm is considered to be of good quality rock without due consideration of its strength and joint spacing.

**12. What is the principle of airlifting for cleaning pile bores? (BP7)**

Airlifting is normally carried out prior to concreting to remove debris and clean the base of pile bores. It essentially acts as an airlift pump by using compressed air. The setup of a typical airlifting operation is as follows: a hollow tube is placed centrally inside the pile bore and a side tube is connected to the end of the tube near pile bottom for the passage of compressed air inside the tube. The upper end of the tube is linked to a discharge tank for the circulation of pumped fluid from pile base.

The efficiency of airlifting operation is dependent on the performance of air compressor. During airlifting, compressed air is piped down the tube and it returns up to the discharge tank carrying it with the fluid. It functions by imparting energy to the fluid and forces the fluid to move vertically upwards. The injected air mixes with the fluid, resulting in the formation of lower unit weight of the combined mixture when compared with surrounding fluid. This hydrostatic pressure forces the fluid/air mixture up to the discharge tanks.

**13. What is the design approach for the spacing of min-piles? (MP1)**

For close spacing of min-piles, it would provide substantial cost savings with the reduction of pile cap size. However, close spacing of piles implies the problem of group effect which tends to reduce the load carrying capacity of each pile member. Notwithstanding this, there is well established rule which govern the minimum spacing of piles, i.e. for friction piles like mini-piles, the centre-to-centre spacing should not be less than the perimeter of the pile.

**14. Should a pipe or groups of bars be adopted as load carrying element of min-piles?**

The design of mini-piles somehow differs from other traditional pile types. For instance, the design of most common pile types is controlled by the external carrying capacity. However, owing to the small cross sectional area, the design of mini-piles is limited by internal carrying capacity. Hence, the choice of suitable load carrying element is of paramount importance in the design of mini-piles.

For steel pipes used as load carrying element, it is of circular cross section with a high radius of gyration. Moreover, it possesses a constant section of modulus in all directions, it serves the properties of excellent column. Bars are suitable when pure axial loading is required in confined situation.

**15. Should engineers consider strain compatibility when designing mini-piles? (MP2)**

In designing the axial capacity of mini-piles, grout may be taken into account in the contribution of axial load capacity. However, the total load capacity of min-piles may not be equivalent to the sum of individual capacity derived from grout and from steel H-section. The reason behind this is that the vertical loads on mini-piles are shared among grout and steel sections based on their Young’s modulus and areas. Basically, in order to comply with strain compatibility criterion, the steel bars and grout will deform as a whole though they possess different stiffness. A case may occur in which the sharing of loads for grout may be too high which cracks the grout section and fails the mini-piles already before the whole pile section could attain the full design load which is assumed to be the sum of individual capacities. Hence, strain compatibility has to be checked in designing the vertical capacity of min-piles [28].

**16. In designing mini-piles, should the strength of grout be neglected during assessment of loading carrying capacity? (MP3)**

In designing min-piles, there are two approaches available:

(i) In the first approach, the axial resistance provided by the grout is neglected and steel bars take up the design loads only. This approach is a conservative one which leads to the use of high strength bars e.g. Dywidag bar. One should note that bending moment is not designed to be taken up by min-piles because of its slender geometry.
(ii) In the second approach, it involves loads to be taken up by both grout and steel bars together. In this way, strain compatibility requirement of grout and steel has to be satisfied.

**17. What are the considerations in determining whether casings should be left in for mini-piles? (MP4)**

Contrary to most of pile design, the design of min-piles are controlled by internal capacity instead of external carrying capacity due to their small cross-sectional area.

There are mainly two reasons to account for designing mini-piles as friction piles:

(i) Due to its high slenderness ratio, a pile of 200mm diameter with 5m long has a shaft area of 100 times greater than cross-sectional area. Therefore, the shaft friction mobilized should be greater than end resistance.
(ii) Settlements of 10%-20% of pile diameter are necessary to mobilize full end bearing capacity, compared with 0.5%-1% of pile diameter to develop maximum shaft resistance.

Left-in casings for mini-piles have the following advantages:

(i) Improve resistance to corrosion of main bars;
(ii) Provide additional restraint against lateral buckling;
(iii) Improve the grout quality by preventing intrusion of groundwater during concreting;
(iv) Prevent occurrence of necking during lifting up of casings during concreting.

## 18. What is the purpose of post-grouting for mini-piles? (MP5)

Post-grouting is normally carried out some time when grout of the initial grouting work has set (e.g. within 24 hours of initial grouting). It helps to increase the bearing capacity of mini-piles by enhancing larger effective pile diameter. Moreover, it improves the behaviour of soils adjacent to grouted piles and minimizes the effect of disturbance caused during construction. In essence, post-grouting helps to improve the bond between soils and grout, thereby enhancing better skin friction between them.

During the process of post-grouting, a tube with a hole at its bottom is lowered into the pile and grout is injected. The mechanism of post-grouting is as follows: the pressurized grout is initially confined by the hardened grout and can hardly get away. Then, it ruptures the grout cover and makes its way to the surrounding soils and into soft regions to develop an interlock with harder soil zones. In order to enhance the pressurized grout to rupture the initial grout depth, a maximum time limit is normally imposed between the time of initial grouting and time of post-grouting to avoid the development of high strength of initial grout. Consequently, the effect of soil disturbance by installation of casings and subsequent lifting up of casings would be lessened significantly.

## 19. What is the purpose of shaft grouting for friction barrettes? (MP5)

For the construction of friction barrettes, some grout pipes are designed at the periphery of the barrettes. Within a short duration (e.g. 24 hours) of concreting of barrettes, the fresh concrete cover is cracked by injecting water. After that, shaft grouting is conducted where the grout travels along the interface between concrete and soil and compacts the surrounding soils which are loosened or disturbed during excavation. The hydraulic fracturing of surrounding soils by grout during the grouting operation generates planes of higher shear strength. The grout would penetrate and improve the strength of soils around the barrettes.

## 20. What is the value of micropiles when compared with bored piles and driven piles?

Micropiles are defined as bored piles with diameters not exceeding 250mm. Micropiles were first constructed in Italy in 1950s and were given the name “root piles”.

Whenever bored piles (larger diameter) and driven piles are considered feasible, they should be more economic than micropiles. Owing to the small size of piles, only small dimension of equipment is needed for construction. Moreover, it can be used to drill through any type of soils, boulders and hard materials.

Micropiles have extensive applications under the following situations:

(i) Underpinning and retrofitting existing structures;
(ii) Locations of limited vertical clearance and small working areas.

**21. How do rock sockets take up loads? (RS1)**

The load transfer mechanism is summarized as follows:

When a socketed foundation is loaded, the resistance is provided by both rock socket wall and the socket base and the load distribution is a function of relative stiffness of foundation concrete and rock mass, socket geometry, socket roughness and strength. At small displacements the rock-socket system behaves in an elastic manner and the load distribution between socket wall and socket end can be obtained from elastic analysis. At displacements beyond 10-15mm, relative displacement occurs between rock and foundation and the socket bond begins to fail. This results in reduction of loads in rock-socket interface and more loads are transferred to the socket end. At further displacements, the interface strength drops to a residual value with total rupture of bond and more loads are then distributed to the socket end.

**22. What is the significance of allowing the usage of combined shaft resistance and end bearing of rock socket to resist loads? (RS2)**

In some design codes, it allows the simultaneous usage of shaft and base resistance of rock sockets to resist loading. Past experience showed that the shaft resistance could be mobilized in rock socket provided that the length of rock socket is less than three times the pile diameter. Such provision eliminates the need for bell-out because higher pile capacity could be derived from the simultaneous usage of shaft and base resistance of rock sockets. Bell-outs are commonly used in piling industry but its effectiveness is doubtful because of some practical issues like the inability of thorough cleaning of pile base.

## Level One (Core FAQs)

### ***Part II: Driven Piles***

#### 1. Is Hiley's formula suitable for clayey soils? (DP1)

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.

#### 2. Is Hiley's formula suitable for estimating capacities all driven H-piles? (DP1)

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.

#### 3. What are the functions of cap block, drive cap and pile cushion in driven piles? (DP2)

Cap block is installed between the hammer end and the drive cap to control the hammer blow in order to protect both the hammer and the pile from damage. When the hammer hits the cap block, it compresses elastically and reduces the peak forces, thereby lengthening the time of hammer blow.

Moreover, it should be capable of transmitting the hammer energy effectively to the piles.

Drive cap is inserted at hammer tip to enhance uniform distribution of hammer energy to the pile. Pile cushion is positioned between the drive cap and the pile top. It intends to protect the pile from driving stress induced during hammer blows. Moreover, it also serves to provide a uniform driving load on top of the pile.

**4. Should light or heavy hammers be used in pile driving? (DP3)**

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.

During driving, the piles are continuously subjected 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.

**5. In driven piles, the allowable set is limited between 25mm and 50mm per 10 blows. Why? (DP4)**

In final set table it is commonly to limit pile set between 25mm/10 blows and 50mm/10 blows. The reason of the provision for lower limit of set value of 25mm/10 blows is to avoid possible damage at pile toe owing to compressive stress reflected at the bottom of pile. The upper limit of 50mm/10 blows is established principally to avoid very heavy hammer impacting at pile head so that the pile head may be damaged owing to excessive compressive stress. These limitations are believed to be applied originally to precast concrete piles.

**6. What is the widespread usage of 45$^o$ load spread rule? (Dp5)**

Under the 45$^o$ load spread rule, when the horizontal distance between the toe level of adjacent piles is smaller than the vertical difference between the piles, additional load is deemed to be added by the pile at lower founding level. Hence, when the pile at lower founding level is within the zone of $45^o$ spreading from the pile at higher elevation, further load checking is required for the lower pile.

This $45^o$ load spread rule is a common practice of foundation engineers and may not be incorporated in foundation codes. There are some situations where this rule may fail. For instance, when two piles at the same foundation level are situated close to each other, they shall have load effect on each other. However, based on $45^o$ load spread rule, it could not address this concern.

**7. Which one is better in driven piles, high hammer/pile weight ratio or low hammer/pile weight ratio? (DP3)**

Boussinesq's closed form solution for a rod fixed at its end and hit on its top by a mass shows that compressive stress in the rod increases with the mass of hammer. On the other hand, a larger relative mass of hammer leads to lower tension stress. When an impact is made on friction pile, the compressive stresses are highest at pile top. When an impact is made on end-bearing pile, the compressive stresses may be highest at the top or the bottom of the pile.

To achieve optimal pile driving operation, the piles should be installed quickly with low blow counts. This can be achieved by heavy hammer but it is uneconomical as it requires higher lifting equipment cost and transportation cost. A lighter hammer appears to be more economical but for the same impact energy as heavy hammer, it requires a greater stroke and impact velocity which may cause damage to pile.

In fact, low hammer/pile weight ratio leads to damage at pile top or cracks along the pile. High hammer/pile weight ratio may cause compressive overstressing at the pile bottom.

**8. Some piling contractors incline to use drop hammer instead of hydraulic hammers in setting of piles. Why? (DP3 & DP4)**

The use of high grade and heavy steel pipes may not warrant the use of drop hammer. For pile driving of Grade 55C steel section, it requires heavier drop hammer with increased drop height so as to comply with penetration resistance at final set. As such, it is possible that such heavy hammer may damage the pile and endangers the worker who takes the final set record.

Though hydraulic hammer is commonly used as standard hammer for driving piles, some piling contractors prefer to employ drop hammers to take final sets. The obvious reason of such practice is that it is convenient for drop hammer to change energy input easily to cater for the range of penetration in final set. On the other hand, the use of hydraulic hammers for setting of piles requires the knowledge of transfer of energy to pile head and hence needs the input from wave equation analysis.

**9. In some codes, they limit the ratio of weight of hammer to weight of pile for pile driving. What is the reason behind this? (DP3)**

When a hammer with initial motion collides with a stationary pile, the transfer of energy is most efficient when the two masses are comparable. That is the reason why some codes limit the ratio of weight of hammer to the weight of pile to be more than 0.5. If the weight of hammer is too low, most of energy during hammer driving is distributed to the hammer and this causes tension induced in hammer and results in inefficient transfer of energy.

**10. Should engineers rely solely on Hiley's formula in the design of H-piles? (DP1)**

About 90% of H-piles adopt Hiley's formula for design. However, this formula is only applicable to pile lengths less than 30m and is suitable for course-grained materials (not suitable to fine-grained soils) as suggested by GEO (1996). In Hiley's formula, by observing the penetration of piles after the hammer impact, the pile capacity could be readily obtained from the response of the impacting force. Therefore, the individual pile capacity could be obtained by this dynamic method.

However, in normal foundation, groups of H-piles are present and the soil foundation may not be able to support these H-piles simultaneously even though individual piles are proven to have sufficient capacity by using dynamic method. In this case, static method should be employed to ascertain if the soil foundation could support these H-piles.

**11. In Hiley's formula for driven piles i.e. R=E/(s+0.5c), why is a coefficient of 0.5 applied for the term elastic deformation of piles and soil? (DP1)**

Hiley's formula is based on the principle of energy conservation in which the energy brought about by hammers during the action of hitting are transferred to piles in ground. When the hammer force and displacement is plotted, the energy absorbed by piles is the area under the curve. Since the curve of elastic component is linear with a positive slope, the area i.e. energy should be the area of triangle (0.5xRxc) where R is reaction force and c is elastic compression due to helmet, piles and soil system. For settlement, it is of horizontal line in force-displacement diagram and hence the energy transferred to pile-soil system is (Rxs).

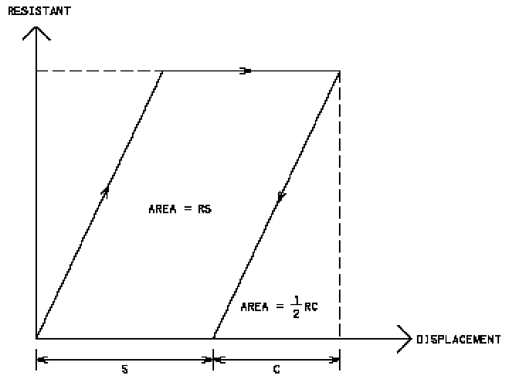



**Fig. Settlement displacement curve for pile driving**

## 12. What is the function of followers in driven H-piles? (DP6)

A follower is an extension between the pile head and the hammer that transfers the blow to the pile in which the pile head cannot be reached by the hammer or is under water .For construction of driven piles, the piling frame and hammer are normally erected on existing ground level but not at the base of pile caps. However, H-piles are designed to be terminated near the base of pile caps. If piles are driven at ground level, a certain length of H-piles is wasted and cut when constructing pile caps. In this connection, pile followers are used so as to save the wasted section of H-piles because followers can be removed during subsequent construction of pile caps.

## 13. What is the difference between hammer efficiency and coefficient of restitution? (DP7)

Hammer efficiency refers to the ratio of kinetic energy of the hammer to the rated energy (or potential energy). In essence, it is undoubtedly that certain energy losses are induced by the hammer itself prior to the actual impact on the driven piles. For instance, these losses may include the misalignment of the hammer, energy losses due to guiding friction, inaccurate dropping height etc…

Coefficient of restitution refers to a value indicating the strain energy during collision regained after the bodies reverting back to their original shapes. If the coefficient of restitution is equal to unity, it means that the collision is elastic and all energy has been returned after the impact action. Hence, this is an index showing the degree the impact action in terms of elasticity.

In mathematical forms,

coefficient of restitution = $-(v_1-v_2)/(u_1-u_2)$
where u=initial velocity and v=final velocity after impact

**14. In pile driving operation, would soils always exhibit an increase in pore water pressure?**

The change in pore water pressure varies in different soils. In loose sands and sandy silts, the pore water pressure increases during pile driving owing to soil densification. The increase in pore water pressure reduces the soil strength. However, after piling operation ceases for a certain period of time, upon dissipation of pore water pressure the soils would result in increased strength by soil “set-up”.

For dense sands, the piling operation cause dilation and increases the pore volume. As such, water may not be fast enough to infiltrate to equalize the pore pressure and this results in a reduction of pore water pressure. Therefore, the apparent increased soil strength is temporary only and it would be reverted back when soil relaxation takes place soon.

**15. Should thin or thick bitumen layer be used to reduce negative skin friction in driven piles? (DP8)**

When piles are driven through an upper layer of granular soils, thick (10mm) bitumen layer would be scrapped off during the driving process. A thin coat of 1mm to 2 mm thick is not likely to flow in storage and to peel off during pie driving. The bitumen can be applied by brushing or sprinkling after heated to a liquid state. In cold weather conditions, it may be difficult to handle hot bitumen. As such, the bitumen could be mixed with solvent to soften the bitumen which should be able to cure rapidly to ensure that the bitumen coat stays on the piles.

For precast piles, a primer could be added to achieve better cohesion of bitumen to the piles.

**16. Should bitumen be applied to the whole section of driven piles?**

In a certain region of H-piles subjected to ground water table fluctuation, painting is sometimes applied on the surface of H-piles because the rise and fall of water table contribute to the corrosion of H-piles. On the other hand, to reduce the effect of additional loads brought about by negative skin friction, bitumen is applied on the pile surface corresponding to the region of soils that has negative skin friction. However, bitumen should not be applied to the whole section of H-piles because it will be unable to derive the designed frictional reaction from soils. Actually, some engineers have reservation of the effectiveness of bitumen slipcoat because the bitumen may get removed during pile driving.

**17. What is the purpose of coating driven piles with bitumen? (DP8)**

Coating driven piles with bitumen serves the following purposes:

(i) It acts as friction reducer and could effective reduce the effect of downdrag.
(ii) It offers protection of steel piles against acid attack from soils.
(iii) It helps to prevent pile corrosion.

**18. What are the possible methods to reduce downdrag on piles? (DP8)**

(A) Coating piles with friction reducer such as bitumen.
(B) Predrill a hole firstly, followed by putting in the pile and subsequent filling of annulus with bentonite slurry.
(C) Drive the piles with an oversize shoe and fill the annulus with bentonite slurry.
(D) Adopt double tube method in which the inner pile takes up the structural load and the outer pile carries the downdrag load.
(E) Preload the soils to accelerate settlement.
(F) Use electro-osmosis to enhance water content around the piles and reduce the friction between pile and soils.
(G) Increase the pile capacity by using larger diameter or longer length so that the effect of downdrag is thereby reduced.

**19. In press-in piling, how can driving shoes help to reduce jacking loads?**

Press-in piling involves the use of hydraulic rams to provide the force to jack the pile into ground. The hydraulic ram is part of machine called "Silent Piler" which uses jacked pile to provide reaction force for jacking.

The use of driving shoes could reduce the jacking load. The presence of driving shoes change the flow of soils around pile tip and this leads to reduced effective horizontal stress on internal pile shaft. As a result, this enhances a reduction in shaft friction which is the major element for making up the jacking load.

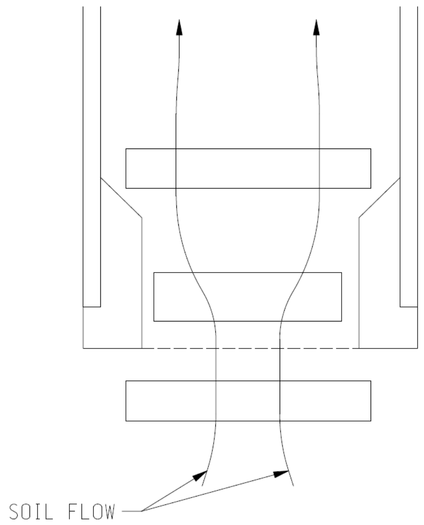


**Fig. Soil flow along driving shoes**

**20. What is the significance of driving sequence of driven piles? (DP9)**

For basement construction, if piles are driven from the centre to the perimeter, there is a tendency of soils to move outwards. Such lateral movement of soil may cause damage to nearby structures and utilities.

However, if piles are driven from the outside perimeter inwards, there are little soil lateral movements. This results in a well-compacted centre with an excess pore water pressure built up to resist the loading of piles. Consequently, shorter pile lengths than the original designed ones may result. However, some time after the pile driving operation, the excess pore water pressure is dissipated and the shorter driven piles may not be able to take up the original design loads. In this situation redriving is required to drive the piles to deeper depths after dissipation of excess pore water pressure.

**21. How does false set occur in pile driving? (DP10)**

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 an apparent temporary increase in driving resistance and strength. However, some time after the pile driving, the dissipation of this negative pore water pressure would reduce the bearing strength in resisting 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 chosen to perform re-driving to check for the false set phenomenon [26].

**22. Why are holes present in steel plates connecting to H-piles?**

There are two kinds of holes present in the steel plate connected to H-piles in the pile cap. The first kind of holes is designed to be filled with welding for better connection with H-piles. The second kind of holes is present to facilitate concreting works of the pile caps. In fact, the void space underneath the steel plate is hardly to be accessed by concrete and these holes provide alternative paths to gain entry into these hidden void.

**23. How can piles be driven through steeply dipping karst surfaces?**

The steep dipping and variable nature of karst surfaces poses problems for installation of driven piles. Very often, the consequences of hard driving piles over steeply-inclined karst are slipping and buckling of piles. To tackle these problems, the following two options are mostly adopted:

(i) Pre-boring is carried out in steep dipping karst surface as this method could penetrate hard layers;
(ii) Reinforcing the end section of driven piles by welding stiffening plates

**24. 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.

**25. There is an old rule that *"the area of a follower should be one-fifth of precast concrete pile"*. Why?**

The rules of wave mechanics suggested that to avoid reflection of stress wave caused by different impedance values, acoustic impedance should be the same for the follower and precast concrete piles. As such, it enhances smoothest driving and prevent follower from bouncing on the head of piles which is undesirable as it may damage the piles and lowers the efficiency of driving.

Acoustic impedance = Elastic Modulus (E) x Area of Pile / wave velocity

Wave velocity (c) = Elastic Modulus / Density of Pile

For normal steel, E=205GPa, c=5,100m/s
For normal concrete, E=30GPa, c=3,800m/s

For same acoustic impedance,

Area of concrete/Area of steel = (205/5,100)/(30/3,800) = 5

# Level One (Core FAQs)

## ***Part III: Pile Tests***

### 1. What is the purpose of conducting load test for piling works? (PLT1)

Pile load test provides information on ultimate bearing capacity but not settlement behavior. In essence, it can determine if the load is taken up by the stratum designed or if the centre of resistance is at the design location in piles as suggested by Robert D. Chellis (1961).

After conducting load tests, the curve of movement of pile head (Settlement against load) and the curve of plastic deformation can be plotted. By subtracting the curve of plastic deformation from the curve of pile head movement at each load, the curve of elastic deformation can be obtained. For piles of end-bearing type unrestrained by friction, the theoretical elastic deformation can be calculated from e=RL/AE where e is elastic deformation, L is pile length, A is area of pile, E is Young's Modulus of pile material and R is the reaction load on pile. By substituting e in the formula, the elastic deformation read from the curve of elastic deformation, L can be obtained which shows the location of the centre of resistance corresponding to that load.

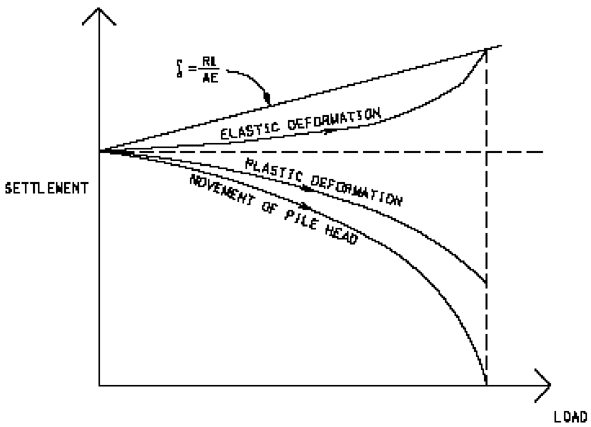



**Fig. Settlement load curve in pile load test**

### 2. What are the meanings of the mathematical terms in failure criteria of pile load test? (PLT2)

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 for 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.

**3. Can dynamic pile test (PDA test) be used to replace pile load test for testing driven pile capacity? (PDA2)**

The PDA system consists of two strain transducers and two accelerometers attached to opposite sides of the pile to measure the strain and acceleration in the pile. The force is computed by multiplying the measured signals from a pair of strain transducers attached near the top of the pile by the pile area and modulus. The velocity measurement is obtained by integrating signals from a pair of accelerometers also attached near the top of the pile.

PDA test is commonly used for the following purposes:

(i) Evaluation of driven pile capacity
Soil resistance along the shaft and at the pile toe generates wave reflection that travel to the top of the driven piles. The time the

reflections arrive at the pile top is a function of their locations along the pile length. The measured force and velocity at the pile top thus provide necessary information to estimate soil resistance and its distribution.

(ii) Measurement of pile stress during driving
Compressive stress at pile top is measured directly from strain transducers.

(iii) Measurement of hammer energy delivered to the piles
The hammer energy delivered to the pile is directly computed as the work done on the pile from the integral of force times displacement ( $\int Fdu$ ) and this can be calculated as force times velocity integrated over time ( $\int Fvdt$ ).

(iv) Determine if pile damage has occurred.
Pile integrity can be checked by inspecting the measurements for early tension returns (caused by pile damage) before the reflection from the pile toe. The lack of such reflections assures a pile with no defects.

There is a growing trend of using PDA test to check driven pile capacity instead of traditional static load test which is more expensive and time-consuming. Some engineers may have reservation on the use of dynamic formula to evaluate driven pile capacity because of the fear of improper selection of quake values and damping factor.

**4. Can both Pile Drive Analyser (PDA) and Pile Integrity Test (PIT) be used for checking pile capacity? (PDA2)**

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.

**5. What is the difference between Case Method and CAPWAP Analysis? (PDA1)**

High Strain Dynamic Testing consists of two main types, namely Dynamic Pile Monitoring and Dynamic Load Testing. Dynamic Pile Monitoring involves the use of PDA to perform real-time evaluation of Case Method pile capacity, hammer energy transfer, driving stresses and pile integrity for every blow count. On the other hand, Dynamic Load Testing is another technique that is evolved from wave equation analysis. CAPWAP Analysis makes use of field measurements obtained by PDA and wave-equation type analytical method to predict pile performance such as static load capacity, pile-soil load transfer characteristics, soil resistance distribution, soil damping and quake values. CAPWAP Analysis is carried out on the PDA data after the test is complete.

**6. What are the limitations of plate load test?**

Plate load test is carried out to check the bearing capacity of foundation soils. The limitations of plate load test are:

(i) It has limited depth of influence. It could only give the bearing capacity of soils with depth up to two times the diameter of plate.
(ii) It may not provide information on the potential for long term consolidation of foundation soils.
(iii) There is scale effect as the size of test plate is smaller than actual foundation.
(iv) To gain access to test position, excavation is carried out which causes significant ground disturbance. The change in ground stress leads to the change of soil properties which the test is planned to investigate.

# Level Two (Advanced FAQs)

## ***Part I: Design of Foundation***

### 1. What are the components in contributing the bearing capacity of shallow foundation? (FD1)

Based on Terzaghi's bearing capacity equation, there are three components in contribution to the bearing capacity:

(i) Surcharge pressure
Foundations are normally not placed directly on the ground level. Instead, they are installed at a depth below the existing ground level. The soil pressure arising from the depth of soils serves as a surcharge imposing a uniform pressure at foundation level.

(ii) Self-weight of soils
The self-weight of soils contribute to the bearing capacity and is represented by $0.5rBN_r$ (r=density of soils).

(iii) Shear strength
The shear strength of soils contributes to the bearing capacity and is represented by $cN_c$.

### 2. What is the difference between pad foundation, strip foundation and raft foundation? (FD2)

Shallow foundation is commonly accepted as foundation with founding level less than 3m from ground surface. In case surface loads or surface conditions could still affect the bearing capacity, the foundation which sits on it is called shallow foundation.

Pad foundation refers to the foundation which is intended for sustaining concentrated loads from a single point load such as structural columns.

Strip foundation is used to support a line of loads such as load-bearing walls. For instance, closely-spaced columns render the use of pad foundation inappropriate and strip foundation may be a better alternative.

Raft foundation consists of a concrete slab which extends over the entire loaded area so that loads from entire structure are spread over a large area leading to a reduction of the stress of foundation soils is reduced. Moreover, raft foundation serves to avoid differential settlement which otherwise would occur if pad or strip foundation is adopted.

**3. What are the differences in function between rock anchors and rock sockets? (FD3)**

Rock anchors are used primarily for resisting uplift forces. On the contrary, rock sockets serve three main purposes:

(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.

Note: 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.

**4. What is the difference between capping beams and ground beams for piles? (FD4)**

Capping beams for piles aim at transferring loads from closely spaced columns or walls into a row of piles. On the other hand, ground beams are beams provided between adjacent pile caps and they perform as compression struts or ties in an attempt to prevent lateral displacement or buckling of piles under uneven distribution of loads on pile caps. Both of them have to be specially designed to cater for differential settlement of piles.

Capping beam performs the same functions as pile caps. However, ground beams are structural elements to connect adjacent pile caps to improve the stability of foundation.

**5. What is the purpose of setting maximum spacing of piles? (FD5)**

One of the factors that affect the distribution of loads from the structures to each pile is the assumption of flexibility of the pile caps in design. A pile cap can be modeled as a flexible or a rigid element based on their relative stiffness. For the pile cap to be assumed as rigid the stiffness of pile cap is infinite relative to that of pile/soil system and the deformations within the cap are not considered owing to its rigidity. On the other hand, for the pile cap to be designed as flexible, internal deformations of pile cap would occur.

In some design guidelines, maximum spacing of piles is specified to limit the length between adjacent piles so that the assumption of rigid pile cap can be justified.

**6. What is the difference between point of virtual fixity and critical length of lateral loading for piles? (FD6)**

Some engineers may get confused about the difference between the two terms i.e. point of virtual fixity and critical length used for piles for resisting lateral loads. For critical length of lateral loading for piles, it refers to a certain depth from the ground level where the piles behave as if it were infinitely long. As such, beyond the critical length, the change in lateral response of piles with increase in pile length will be negligible [26].

Point of virtual fixity refers to a certain dept below ground surface where the piles are fixed without movement under loads. The depth to the point of fixity is useful in assessing the buckling loads of piles. It is obvious that the depth to the point of virtual fixity should be smaller than the critical length of piles.

**7. In modeling a nonrigid mat foundation by using elastic springs, should a uniform modulus of subgrade reaction be used along the whole base of mat? (FD7)**

By using a bed of springs to simulate the flexible behaviour of mat subject to loads, care should be taken in selection of the modulus of subgrade reaction. In fact, the modulus of subgrade reaction depends on many factors like the width of the mat, the shape of the mat, the depth of founding level of the mat etc. In particular, the modulus of subgrade reaction is smaller at the center while it is larger near the mat's edges. If a constant modulus of subgrade reaction is adopted throughout the width of the mat, then a more or less uniform settlement will result when subject to a uniform load. However, the actual behaviour is that settlement in the center is higher than that at side edges. Consequently, it leads to an underestimation of bending moment by 18% to 25% as suggested by Donald P. Coduto (1994).

In general, a constant value of modulus of subgrade reaction is normally applied for structure with a rigid superstructure and the rigid foundation. However, a variable modulus of subgrade reaction is adopted instead for non-rigid superstructure and non-dominance of foundation rigidity to account for the effect of pressure bulbs.

**8. Which type of pile cap transfers loads equally to piles, flexible pile cap or rigid pile cap? (FD8)**

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.

**9. 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.

**10. How should the piles be arranged in a pile cap to reduce bending moment induced in piles?**

Consider that piles are designed to intersect at a single common point in a pile cap. The resultant reactions would pass through the point of intersection in the pile cap. This type of arrangement does not involve any bending moment induced if the horizontal loads pass through this point. However, in real life situation, the piling system is expected to resist a combination of vertical loads, horizontal loads and bending moment. To counteract bending moment, the pile cap about the point of intersection is rotated so that significant amount of bending moment is induced in piles and pure axial forces in piles can hardly generate a counteracting moment based on one single intersection point [53].

However, if the piles are arranged in such a way that there are at least two separated points of intersection in the pile cap, the amount of flexural stresses induced in piles is significantly reduced.

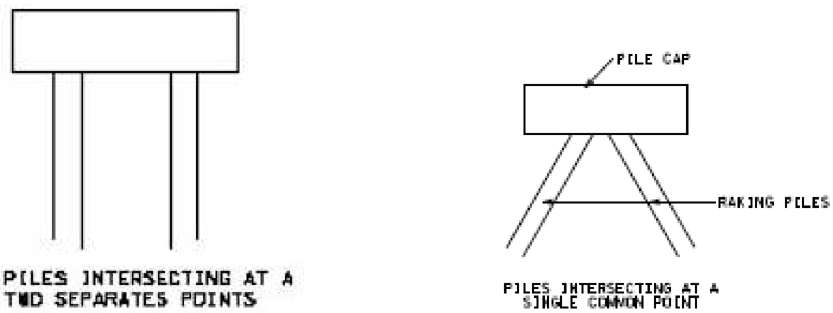



**Fig. Different arrangement of piles in pile cap**

**11. What are the differences between pinned bases and fixed bases? (FD9)**

When structures like portal frames are connected to the base foundation, engineers have to decide the degree of fixity for the connection. In general, the two common design options are pinned bases and fixed bases. Pinned bases have the advantage that the design of foundation is made simple so that some cost savings may result. However, fixed bases design provides additional rigidity and stiffening to the structures and the stability of the structures can be enhanced. Therefore, the use of fixed bases helps to improve the structural performance of the structures [41].

**12. Do edge piles take up same loadings as central piles in rigid cap? (FD10)**

Due 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 a uniform load. 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, raking 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 raking edge piles.

There are several choices regarding the design to tackle the uneven distribution of loads. The first one involves the lengthening of side piles to stabilize the piles under high loads. However, the increased length of outer piles tends to attract more loads and this seems not to be a good solution. The other way out is to lengthen the central piles aiming at getting more loads and this evens out the load distribution among the piles [26].

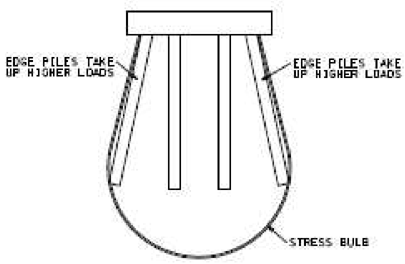



**Fig. Stress bulb effect on load distribution in piles**

**13. Which one is a better choice, a large diameter piles or a system of several smaller piles with the same load capacity?**

The choice of a large diameter pile suffers from the disadvantage that serious consequences would occur in case there is setting out error of the pile. Moreover, in terms of cost consideration, for the same load capacity the cost of a group of small diameter piles is generally lower than that of a large diameter pile. On the other hand, for small diameter piles i.e. mini-piles, they are advantageous in site locations with limited headroom and space. In addition, in some structures with only a few piles, it is uneconomic because of its high mobilization cost. Reference is made to Dr. Edmund C Hambly (1979).

**14. In designing the lateral resistance of piles, should engineers only use the earth pressure against pile caps only?**

In some design lateral loads are assumed to be resisted by earth pressure exerted against the side of pile caps only. However, it is demonstrated that the soil resistance of pile lengths do contribute a substantial part of lateral resistance. Therefore, in designing lateral resistance of piles, earth pressure exerted on piles should also be taken into consideration.

In analysis of lateral resistance provided by soils, a series of soil springs are adopted with modulus of reaction kept constant or varying with depth. The normal practice of using a constant modulus of reaction for soils is incorrect because it overestimates the maximum reaction force and underestimates the maximum bending moment. To obtain the profile of modulus of subgrade reaction, pressuremeter tests shall be conducted in boreholes in site investigation. Reference is made to Bryan Leach (1980).

**15. How does the pile installation method affect the load carrying capacity of piles? (FD10)**

The construction of piles by driving method causes an increase in density of the surrounding soils. Hence, for loose soils this results in improved compaction of soils between the piles. The sum of the capacities of all piles as a whole is generally greater than the sum of individual pile capacities provided that the effect of pile spacing is not taken into account. However, for bored piles the boring operation induces considerable stress relief and this causes a substantial reduction in shear strength of soils.

**16. What are the functions of different reinforcement in a typical pile cap? (FD11)**

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

**17. What are the head details of H-piles under compression and subject to bending moment? (FD11)**

For steel sections referred to in BS5950, universal bearing pile is characterized by having equal flange and web thickness while universal column has different flange and web thickness. Universal columns can also be used as bearing piles.

In the design of the head details of H-piles, there are three typical cases to be considered, namely compression piles, tension piles and piles with bending moment at the head in addition to tension or compression. The design of these piles recommended by G. M. Cornfield (1968) is listed below:

(i) Compression piles
For this type of piles, H-piles should be embedded 150mm in concrete pile

caps and it is not necessary to use any dowels and capping plates in their connection.

(ii) Tension piles
A number of hook-ended bars are welded to the top of H-piles.

(iii) Piles with bending moment at their head (tension or compression)
The depth of embedment of piles into pile caps is substantially increased and loads are transferred by horizontal bars welded to piles' flanges.

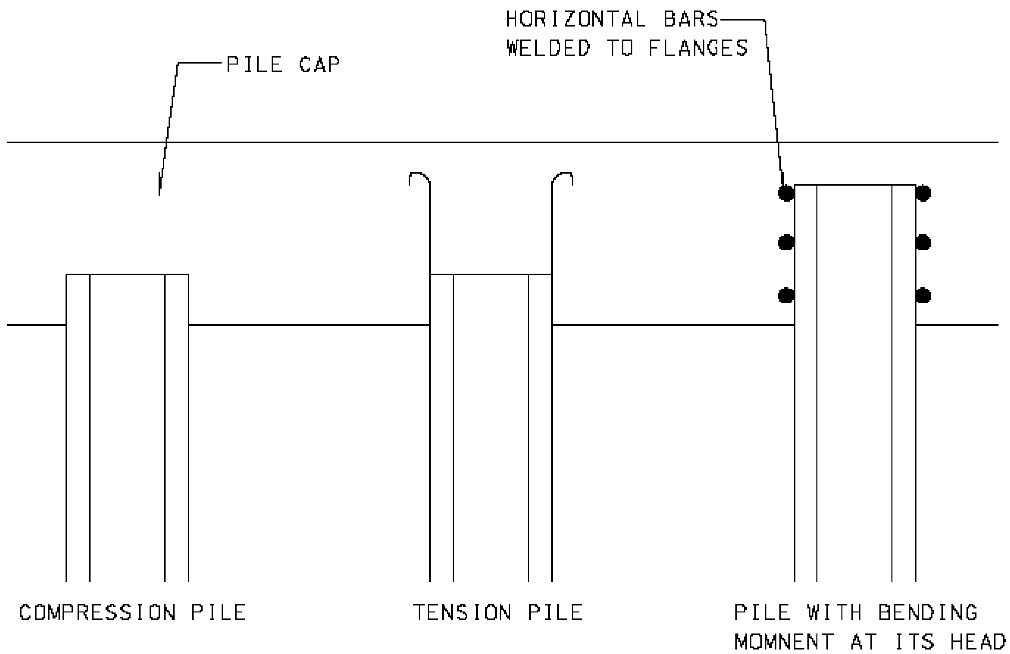



**Fig. Different arrangement of pile head details**

### 18. It is not necessary to design nominal reinforcement to piles. Is it true?

In BS8110 and BS5400 Pt.4, they require the provision of nominal reinforcement for columns. However, for pile design the requirement of nominal reinforcement may not be necessary. Firstly, as piles are located underground, the occurrence of unexpected loads to piles is seldom. Secondly, shear failure of piles is considered not critical to the structure due to severe collision. Moreover, the failure of piles by buckling due to fire is unlikely because fire is rarely ignited underground.

However, the suggestion of provision of nominal reinforcement to cater for seismic effect may be justified. Reference is made to J P Tyson (1995).

**19. How to measure rock during piling operation?**

There is no strict rule in governing the measure of rock encountered in piling operation. There are two common practices in measuring rock during piling:

(i) Measure the quantity of obstruction taken out from the drilled out;
(ii) Firstly, it is assumed that the rock surface is uniform. Based on this assumption, measure the obstruction level by using a tape.

**20. What are the advantages of using top-down approach in basement construction?**

The advantages of top-down approach are listed below:

(i) The structures above ground can be carried out simultaneously with the structures below ground. This greatly reduces the time for construction.
(ii) By using this approach, settlement can be reduced.
(iii) Since the permanent columns and slabs can be utilized to support loadings during construction, it saves the cost of formwork.

Note: Top-down approach means construction of basement is carried out from ground level downwards.

**21. What are the methods to tackle negative skin friction? (FD12)**

(i) Use slender pile sections (e.g. H-pile or precast pile) because smaller pile area when subject to the same working load would produce higher deformation, thus increasing the relative downward movement of piles.
(ii) In a certain region of H-piles for ground water table fluctuation, painting is applied on the surface of H-piles because the rise and fall of water table contribute to the corrosion of H-piles. On the other hand, to reduce the effect of additional loads brought about by negative skin friction, bitumen is applied on the pile surface corresponding to the region of soils that has negative skin friction. However, bitumen

should not be applied to the whole section of H-piles because it would be unable to derive the designed frictional reaction from soils.

(iv) Design the piles as end-bearing so that they can take up more load.

**22. How do fixed and pinned connections between piles and pile caps affect the load carrying capacity of piles?**

The type of connection between piles and pile caps affects the load carrying capacity of pile groups. The fixity of pile head into pile cap, instead of pinning into pile cap, enhances higher lateral stiffness of the pile groups. For instance, for the same deflections, a cap with fixed connected piles can sustain far more loads than that of pinned connected piles. To satisfy the criterion of fixed connection, the minimum embedded length of piles into pile caps should be at least two times the diameter of piles.

Moreover, the fixed connection of piles at pile caps allows significant bending moment to be transmitted through the connections when compared with pinned connections.

**23. Is critical depth of piles a fallacy? (FD6)**

The critical depth of piles are normally assumed as 10-20 pile diameter deep and is the depth beyond which the resistance is constant and is equal to respective value at critical depth.

The critical depth is a fallacy which comes from the failure to interpret the results of full and model-scale pile tests. In full-scale test, the neglect of presence of residual loads renders a measured load distribution to be linear below the so called “critical depth”. Residual loads refer to loads that are induced in piles during and after installation of piles.

**24. 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.

**25. What is the difference between compaction grouting and fracture grouting? (FD13)**

Grouting can be implemented in two common modes, namely compaction grouting and fracture grouting. For compaction grouting, high viscosity grout is commonly used for injection into soils. Upon reaching the soils, the grout would not penetrate into soil spaces. Instead it forms a spherical bulb and remains as a homogeneous mass. The formation of bulb displaces the nearby soils.

Fracture grouting involves the use of low viscosity grout. Upon injection, the grout would split open the ground by hydraulic fracturing and penetrate into the fractures. Similarly, soils are displaced during the process.

# Level Two (Advanced FAQs)

## ***Part II: Miscellaneous***

**1. During concreting of diaphragm walls, three tremie pipes are used in one time. However, only one concrete truck is available. How should the concreting works be carried out? (DW1)**

The most ideal situation is to supply each tremie pipe with a single concrete truck. However, if only one concrete truck is available, all the fresh concrete in the truck should not be placed in one single tremie pipe. With all fresh concrete placed in one single tremie pipe while the others left void, then due to the huge supply of concrete to the tremie pipe, a small concrete hump may form at the base of the tremie pipe and it is likely that it may collapse and trap the slurry inside the diaphragm walls. Therefore, the fresh concrete should be evenly shared among the tremie pipes to avoid such occurrence.

**2. What is the significance of quality of bentonite slurry in the construction of diaphragm walls? (DW2)**

The quality of slurry plays an important role in the quality of diaphragm walls. Firstly, if a thick slurry cake is formed in the interface between slurry and in-situ soil, it has a tendency to fall off during concreting works and it mixes with freshly placed concrete. Moreover, large thickness of slurry cake would reduce the concrete cover and affect the future durability performance of diaphragm walls.

**3. When are prestressed tiebacks used in sheet piling works? (SP1)**

The use of prestressed tiebacks gets rid of the need of interior bracing. Prestressed tiebacks are anchored into rock or granular soils and excavation can be conducted by using powerful shovel instead of using hand excavation or other small excavators. It provides less restraint and allows free movement for excavation.

**4. In seismic liquefaction, what is the difference of pile failures mechanism between lateral spreading and buckling?**

Most of design codes assume that pile fails during strong earthquake by lateral spreading. Lateral spreading is based on bending mechanism where the inertia and slope movement causes bending in piles. In essence, piles are considered as beams which are subjected to lateral loads such as slope movement leading to pile failure.

Piles are slender columns with lateral support from foundation soils. When the length of pile increases, the buckling loads decrease with the square of pile length. For buckling failure, soils around the piles lose the confining stress during earthquake and can hardly provide lateral support to piles. As such, the pile serves as an unsupported column with axial instability. It will buckle sideways in the direction of least bending stiffness under axial load.

**5. What are the reasons in observed settlements in rockfill foundation?**

Compression of rockfill is normally caused by a reduction in dimension of fill and by rearrangement of particles into closer packing.

When the rockfill are saturated, the strength of rock would be reduced accordingly. In fact, wetting of rock surfaces does not reduce the coefficient of sliding friction between rockfills. Considerable settlement may result not from the lubricating effect of water but from a reduction of rock strength at its point of contact. The contact points would then be crushed under intergranular force and the contact area increases until contact pressure is less than the strength of rockfill.

Rockfill with sharp corners proved to be more liable to settlement than those of well-rounded.

To minimize settlement of rockfill, the intergranular force should be reduced and this is achieved by grading the size of rock particles such that there is minimum amount of voids and hence a maximum amount of particle contacts. To avoid particle rearrangement under future loading, the rockfill should be properly compacted with earth-moving machinery.

**6. Should bentonite be added to improve the stability of grout? (G1)**

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. Why can't normal Reversed Circulation Drills function in shallow rock conditions? (CM1)**

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.

**8. Can down the hole hammer function below water table? (CM2)**

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.

**9. What is the purpose of shaft grouting of deep foundations? (G2)**

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.

**10. 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.

**11. Should compaction be carried out to freshly-placed concrete piles? (PC1)**

In normal practice, reliance is placed on the self-compaction of specially designed concrete mixes to achieve adequate compaction. The use of vibrating devices like poker vibrators is seldom adopted for the compaction of concrete piles. In fact, other than the consideration of the impracticality in using vibrating device in long piles, there is serious concern about the possible occurrence of aggregate interlock which poses difficulty during casing extraction [64]. In the worst scenarios, the temporary casings together with reinforcement cages are extracted during the lifting up of pile casings. This is another reason which accounts for not using vibrating machines for piles with casing extraction.

**12. Is the quality of concrete impaired by free-falling concrete placement method in bored piles? (PC2)**

Based on the research by STS Consultants Ltd. [57], it was found that concrete placed by free falling below 120 feet would not suffer from the problem of segregation and the strength of concrete would not be detrimentally impaired provided that the piles' bore and base are dry and free of debris. Moreover, it is presumed in the past that during free falling of fresh concrete into the pile bores the hitting of falling concrete in the reinforcement cage causes segregation. However, in accordance with the experimental results of STS Consultants Ltd. [57], the striking of reinforcement cage by fresh concrete does not have significant effect on the strength of concrete

In addition, for long bored piles, it is impractical to conduct vibration to concrete. For concrete placed by free falling method, the impact action arising from free falling is assumed to induce adequate vibration. On the other hand, concrete placed by tremie method appears to be lack of vibration and this may affect the strength and integrity of concrete. The research results showed that the strength of vibrated concrete was slightly higher than unvibrated concrete. Vibration proved to have added advantage to concrete strength but not essential to achieve the design pile strength.

**13. Is pile tip cover necessary for rock-socketed H-piles?**

In current practice concrete cover is usually provided at the pile tips of pre-bored H-piles socketed in rock. The purpose of such arrangement is to avoid the potential occurrence of corrosion to H-piles in case concrete cover is not designed at pile tips. However, recent field and laboratory observations had reservation of this viewpoint [45]. In case H-piles are designed to be placed directly on top of rock surface, it provides the tip resistance to limit the pile movement in the event of bond rupture between grout and H-piles. As such, some contractors may choose to tamp the H-piles by using drop hammers to ensure the H-piles are founded directly on top of rock surface. Practically speaking, it poses difficulties during the process of tamping because there is a chance of possible buckling of long H-piles when too much energy is provided to the piles.

**14. In deep excavation, adjacent ground water table is drawn down which may affect the settlement of nearby buildings. What is the remedial proposal to rectify the situation? (DE1)**

One of the methods to control settlement of nearby buildings due to excavation work is by recharging. Water collected in wells in deep excavation is put back to the top of excavation in order to raise the drawn-down water table. The location of recharge should be properly selected to ensure the soil is sufficiently permeable to transfer the pumped water back near the affected buildings.

## 15. What are the problems associated with prestressed concrete piles (Daido)? (PCP1)

The origin of Daido piles comes from Japan where these prestressed concrete piles are used as replacement plies. Holes are pre-formed in the ground and Daido piles are placed inside these pre-formed holes with subsequent grouting of void space between the piles and adjacent ground. However, in Hong Kong Daido piles are constructed by driving into ground by hammers instead of the originally designed replacement method. Since the installation method of Daido piles is changed, construction problems like deformation of pile tip shoes, crushing of concrete at pile tip etc. occur. Reference is made to B. W. Choy (1993).

# 7. Module Six: Roadworks

## Objectives

| Element | Description | Objective No. |
|---|---|---|
| **Bituminous Road** | | |
| Road Structure | Roadbase | RS1 |
| | Friction course | RS2 |
| | Wearing course | RS3 |
| | "Mortar mechanism" and "stone contact mechanism" | RS4 |
| | SMA | RS5 |
| | Anti-skid dressing | RS6 |
| Bituminous Materials | Tar | BM1 |
| | Aggregates, filler and binder | BM2 |
| | Air void | BM3 |
| Construction | Temperature | C1 |
| | Compaction | C2 |
| | Bleeding | C3 |
| | Prime coat | C4 |
| | Tack Coat | C5 |
| **Concrete Road** | | |
| Construction | Slump | C1 |
| | Time to saw contraction joints | C2 |
| | Box-out | C3 |
| Pavement Structure | Reinforcement | PS1 |
| | Joints | PS2 |
| | Sub-base | PS3 |
| | Priming coat | PS4 |
| | Capping layer | PS5 |
| | Separation membrane | PS6 |
| | Lean concrete base | PS7 |
| | Corners | PS8 |
| Test | Ten percent fines value | T1 |
| | Surface regularity test and sand patch test | T2 |
| **Paving Blocks** | | |
| Structure | Load transfer mechanism | S1 |

| Element | Description | Objective No. |
|---|---|---|
| | Sand layer | S2 |
| | Edge courses | S3 |
| | Jointing sand | S4 |
| **Road Joints** | | |
| Road Joints | Joint sealant | RJ1 |
| | Plastic sleeve in dowel bars | RJ2 |
| | Longitudinal joints | RJ3 |
| | Unsealed contraction joints | RJ4 |
| | Tie bars and dowel bars | RJ5 |
| | Keyway joint | RJ6 |
| **Pavement Design** | | |
| Asphalt Mix Design | Principle | AMD1 |
| | Marshall Mix Design | AMD2 |
| | Marshall stability and flow test | AMD3 |
| | Air voids | AMD4 |
| | Recipe Approach | AMD5 |
| Steel Reinforcement | High-yield steel or mild steel | SR1 |
| | Direction of placement | SR2 |
| Road Formation | California Bearing Ratio | RF1 |
| | Subsoil drains | RF2 |
| Road Stiffness | Benkelman Beam Test | RS1 |
| | Falling Weight Deflectometer Test | RS2 |
| **Road Furniture** | | |
| Concrete Profile Barriers | Corrugation | CPB1 |
| | Position | CPB2 |
| Barrier Types | Safety fence and safety barrier | BT1 |
| | Tensioned corrugated beam and untensioned corrugated beam | BT2 |
| Vehicular Parapet | Stiffness | VP1 |
| Other Road Furniture | Oil interceptors | ORF1 |
| | Kerb overflow weirs | ORF2 |
| | Sag gully and on-grade gully | ORF3 |
| | Kerbs | ORF4 |
| | Road studs | ORF5 |
| | Road marking | ORF6 |

# Level One (Core FAQs)

## *Part I: Bituminous Road*

### 1. What are the differences in function between roadbase and basecourse in flexible carriageway? (RS1)

Roadbase is the most important structural layer in bituminous pavement. It is designed to take up the function of distributing the traffic loads so as not to exceed the bearing capacity of subgrade. In addition, it helps to provide sufficient resistance to fatigue under cyclic loads and to offer a higher stiffness for the pavement structure.

However, the basecourse is normally provided to give a well-prepared and even surface for the laying on wearing course. Regarding the load distribution function, it also helps to spread traffic loads to roadbase but this is not the major function of basecourse.

### 2. What are "mortar mechanism" and "stone contact mechanism" in bituminous materials? (RS4)

"Stone contact mechanism" applies to well graded aggregates coated with bitumen (e.g. dense bitumen macadam) where the traffic loads on bituminous roads are resisted by stone-to-stone contact and by interlocking and frictional forces between the aggregates. It is essential to adopt aggregates with a high crushing strength. The bitumen coatings on the surface of aggregates merely serve to cement the aggregates together.

"Mortar mechanism" involves the distribution of loads within the mortar for gap-graded aggregates (e.g. hot rolled asphalt). The mortar has to possess high stiffness to prevent excessive deformation under severe traffic loads. It is common practice to introduce some filler to stiffen the bitumen.

### 3. What is the importance of friction course to expressways and high speed roads? (RS2)

Friction course, which is known as porous asphalt, is often used as surface material in high speed roads and expressway. It is porous in nature which allows for speedy drainage of surface water. Road safety can be improved because it reduces the chance of hydroplaning/aquaplaning (i.e. there is a layer of water between the tires of the vehicle and the road surface. It may lead to the loss of traction and thus prevent the vehicle from responding to control inputs such as steering, braking or accelerating). It also decreases the splash and spray from vehicles in wet weather. It serves as a drainage channel for water to flow beneath the pavement surface.

Friction course has larger texture depth and this enhance improved skid resistance for vehicles traveling at high speed. It also has other benefits like the reduction of noise generated by vehicle tyres.

**4. Why do some countries have reservation on the use of friction course as bituminous surface layers? (RS2)**

Owing to the porous nature of friction course, its deterioration rate is faster than conventional bituminous materials and the durability of friction course is a main concern form users. The service life of friction course is reported to be around 8-10 years.

The main durability problem of friction course is associated with raveling of friction course and stripping of underlying layers. Raveling of friction course occurs as a result of lack of cohesion between aggregates. Stripping of underlying layers is attributed by inadequate drainage of water through the friction course. With the use of polymer in friction course, it permits the use of higher air void (allows for better drainage) and higher binder content.

**5. What is the purpose of tar in bituminous materials? (BM1)**

Tar is commonly incorporated in bituminous materials because of the following reasons:

(i) Blending of tar with bitumen possesses better binding performance with roadstone than bitumen.
(ii) Resistance to fuel oil erosion is high. Tar is used in roads where there is frequent spillage of fuel from vehicles.

**6. What are the different functions of aggregates, filler and binder in bituminous pavement? (BM2)**

In bituminous materials, course aggregates perform the bulking action of the mixture and contributes to the stability of resulting mix. Fine aggregates form the major proportion of mortar.

Filler: it stiffens and strengthens the binder.

Binder: cements the whole mixture together and provides waterproofing.

**7. What is the importance of air void content in bituminous pavements? (BM3)**

The air void content of bituminous materials is an important control parameter for the quality of bitumen being laid and compacted. If the air void content is too high, it allows for intrusion of air and water. Moreover, it also increases the rate of hardening of binders which produce premature embrittlement of pavements. In addition, too high a void content will also lead to differential compaction subject to traffic loads and result in formation of ruts and grooves along the wheel track.

However, a minimum amount of air void should be maintained to avoid instability during compaction process and to provide space for bitumen flow in long-term consolidation under traffic loads. A sufficient amount of air voids should be designed to make room for expansion of binder in summer and compaction by road traffic as suggested by National Association of Australian State Road Authorities (1968), otherwise bleeding and loss of stability may occur and the pavement will deform readily under severe loads.

**8. In General Specification for Civil Engineering Works (1992 Edition), it specifies the temperature requirements for bituminous material during and after mixing. What is the reason behind this? (T1)**

Temperature is one of the factors that govern the compaction of bituminous material and the air void content is found to decrease with an increase in compaction temperature. This phenomenon is explained by the viscosity-temperature relations: the higher is the viscosity of binders, the greater is the resistance to compaction. Therefore, in normal contract for bituminous laying, the temperature requirements for bituminous material during and after mixing are specified.

**9. How does wearing course provide skid resistance? (RS3)**

The skid resistance of wearing course in a bituminous pavement is contributed by the macrotexture (i.e. the general surface roughness) and the microtexture (i.e. the protruding from chippings) of the wearing course [38]. These two factors affect the skid resistance of flexible carriage in different situations. For instance, when the carriageway is designed as a high-speed road, the tiny channels among the macrotexture help to drain rainwater to the side of the road and avoid the occurrence of aquaplaning. In low speed roads the microtexture has particular significance in providing skid resistance by gripping the car tyres to the road surface.

**10. What is the optimum binder content in bituminous pavement? (BM2)**

The amount of binder to be added to a bituminous mixture cannot be too excessive or too little. The principle of designing the optimum amount of binder content is to include sufficient amount of binder so that the aggregates are fully coated with bitumen and the voids within the bituminous material are sealed up. As such, the durability of the bituminous pavement can be enhanced by the impermeability achieved. Moreover, a minimum amount of binder is essential to prevent the aggregates from being pulled out by the abrasive actions of moving vehicles on the carriageway.

However, the binder content cannot be too high because it would result in the instability of the bituminous pavement. In essence, the resistance to deformation of bituminous pavement under traffic load is reduced by the inclusion of excessive binder content.

**11. How would high temperature affect the laying bituminous pavement? (T1)**

In general, bituminous materials are also broadly classified into two types, namely bitumen macadams and hot-rolled asphalts. During compaction, the increase of temperature causes the reduction of viscosity of binder. The binder acts as a lubricant among aggregates particles because it is mobile in a fluid state under high temperatures. The internal resistance between the bituminous materials is drastically reduced resulting in the formation of a mixture with better aggregate interlock.

Bitumen macadams mainly contain continuously graded aggregates. Compaction of this type of bituminous material is eased with an increase of mix temperature as the lubricating effect of reduced viscosity of binder helps in the rearrangement of aggregates.

The aggregate of hot-rolled asphalt are not well graded. With a rise in mixing temperature, the binder will stay unset and the mixture has little resistance to compaction [38].

**12. What is the desirable compacted thickness of bituminous pavement? (C2)**

The choice of compacted thickness is closely related to the nominal maximum size of aggregates of bituminous materials. Based on the recommendation by Dr. Robert N. Hunter [38], the rule of thumb is that the compacted layer thickness should exceed 2.5 times the maximum size of aggregate. If the layer thickness is less than 1.5 times the nominal maximum size of aggregates, the mechanical properties of bituminous material is impaired by the possible crushing of larger sizes of aggregates. Hence, controlled thickness of compaction of bituminous material should be clearly stated in works specification [38].

**13. What is the purpose of paving bituminous surfacing over concrete structures?**

The use of bituminous surfacing over concrete structures (e.g. existing concrete roads) is widespread to improve the skid-resistance and the general appearance of roads on one hand, and to avoid the pre-mature failure of concrete surface by frost spalling in cold countries on the other hand.

In designing the bituminous surfacing over concrete, there are several areas to which engineers should pay attention. Firstly, the laying of thin bituminous material over the joints or existing cracks of concrete structure would lead to reflective cracking because the thermal movement of concrete induces swift formation of cracks in bituminous surfacing. Past research demonstrated that with the adoption of minimum thickness of 100mm bituminous surfacing the occurrence of reflective cracks would be delayed when compared with the use of thinner surfacing. Secondly, sufficient adhesion between concrete and bituminous surfacing has to be achieved. Therefore, it is recommended to apply a layer of tack coat on the concrete surface to promote bonding.

**14. How do paver, steel-wheeled roller and pneumatic tire rollers carry out compaction? (C2)**

Paver, steel-wheeled roller and pneumatic tire roller compact bituminous material by using the following principles:

(i) The static weight of the paving machines exerts loads on the bituminous material and compresses the material directly beneath the machine. The compacting effort increases with the period of contact and larger machine weight.

(ii) Compaction is brought about by the generation of shear stress between the compressed bituminous material under the machine and the adjacent uncompressed bitumen.

**15. What is the significance of bleeding in bituminous pavement? (C3)**

Bleeding occurs in bituminous pavement when a film of asphalt binder appears on road surface. Insufficient air void is a cause of bleeding in which there is insufficient room for asphalt to expand in hot weather and it forces its way to expand to pavement surface. Too much asphalt binder in bituminous material is also a common cause of bleeding. Bleeding is an irreversible process (the bleeded asphalt on pavement surface would not withdraw in winter) so that the amount of asphalt binder on pavement surface increases with time.

**16. What are the causes of longitudinal cracks and transverse cracks in bituminous pavement?**

Longitudinal cracks in bituminous pavement are usually caused by fatigue failure under repeated traffic loading. In thin pavements, cracking starts at the bottom of the bituminous layer where the tensile stress is the highest and then it spreads to the surface as one or more longitudinal cracks. In thick pavements, the cracks usually commence from the top because of high localized tensile stresses from tire-pavement interaction. After repeated loading, the longitudinal cracks develop into a pattern similar to the back of an alligator.

Transverse cracks are usually formed as a result of thermal movement. It may occur because of shrinkage of the bituminous surface due to low temperatures or asphalt binder hardening.

**17. What is the function of prime coat in bituminous pavement? (C4)**

The principal function of prime coat in bituminous pavement is to protect the subgrade from moisture and weathering. Since the presence of moisture affects the strength of subgrade, the prevention of water entry during construction is essential to avoid the failure of the pavement. In cold countries, by getting rid of moisture from subgrade, the danger of frost heave can be minimized.

Prime coat is an asphalt which, when applied evenly to the surface of sub-base or subgrade, serves to seal the surface to hinder the penetration of moisture into subgrade. Vehicular traffic should be avoided on the surface sprayed with prime coat because the traction and tearing action of vehicles would damage this asphalt layer.

**18. Should emulsified asphalts or cutback asphalts be selected as tack coat in bituminous roadworks? (C5)**

Emulsified asphalt is a suspension of asphalt in water by using an emulsifying agent which imposes an electric charge on asphalt particles so that they will join and cement together. Cutback asphalt is simply asphalt dissolved in petroleum. The purpose of adding emulsifying agent in water or petroleum is to reduce viscosity of asphalt in low temperatures.

The colour of emulsion for tack coat is brown initially during the time of application. Later, the colour is changed to black when the asphalt starts to stick to the surrounding and it is described as “break”. For emulsified asphalts, when water has all evaporated, the emulsion is said to have “set”. Cutback emulsion is described to have been “cured” when the solvent has evaporated. There are several problems associated with cutback asphalts:

(i) Emulsified asphalt can be diluted with water so that a low application rate could be achieved.
(ii) The evaporation of petroleum into atmosphere for cutback asphalt poses environmental problem.
(iii) The cost of production of petroleum is higher than that of emulsifying agent and water.

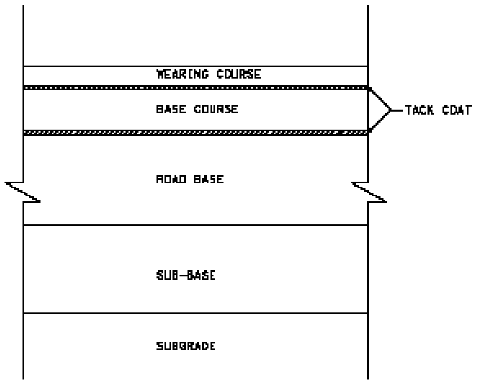



**Fig. Position of application of tack coat**

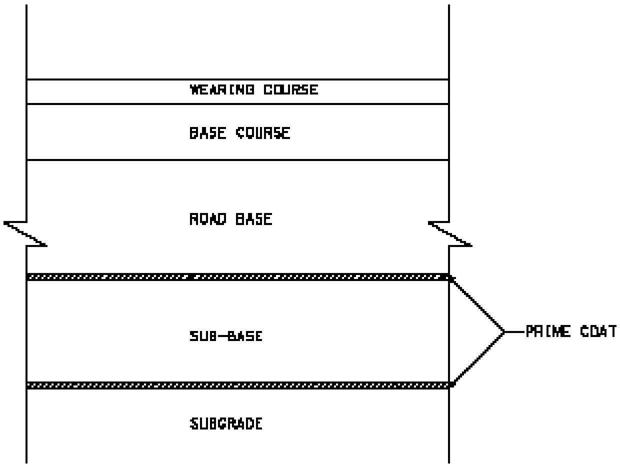



**Fig. Position of application of prime coat**

### 19. Why is it necessary to ensure cleanliness of bituminous pavement before applying tack coat? (C5)

Surface preparation of bituminous pavement is essential in proper application of tack coat. The entire surface has to be cleared of debris, dust and soils. Otherwise, the tack coat would stick to the debris left on the pavement instead of adhering to the pavement. When delivery or placement equipment is driven over tack coat, it tends to stick to the tire's of the equipment instead of pavement.

### 20. What are the differences between anionic emulsions and cationic emulsions?

Bitumen emulsions consist of particles of bitumen dispersed in water by using emulsifying agent. When the emulsion breaks, it represents a change from a liquid to a coherent film with bitumen particles coagulating together. The sign of breaking is the change of colour from brown to black as the colour of emulsion and bitumen is brown and black respectively.

There are in common two broad types of emulsions, namely anionic emulsions and cationic emulsions. The breaking of anionic emulsions is dependent on the evaporation of water from bitumen emulsion. As such, it poses difficulty in wet weather condition. However, for cationic emulsions, instead of relying on the evaporation of water the breaking is achieved by chemical coagulation. Hence, cationic emulsions are particularly useful in wet weather conditions [66].

**21. Why are stone mastic asphalt used in heavily trafficked roads? (RS5)**

Stone mastic asphalt (SMA) developed in Germany in late sixties. It is characterized by having a high proportion of coarse aggregates that interlock to form a strong aggregate skeleton. Typical SMA composition consists of 70-80% coarse aggregates, 10% filler and 6% binder. The concept of SMA design is that severe traffic loads is carried by the stone skeleton, and the mastic (a combination of fine aggregates, filler and binder) is introduced in the mix to fill out the remaining void spaces with an aim in achieving durability. To ensure that the course aggregate stone contact function, it is essential to prevent immediate sizes of aggregate to hold course aggregates apart.

Owing to high binder content, drainage inhibitor is added to prevent binder drainage during transport and placing.

In essence, SMA provides a textured, durable and rut resistant wearing course and hence they are commonly used in heavily trafficked roads with high traffic stress.

**22. Why is it preferable for SMA layer to be cooled below 40$^o$ before opening to traffic? (RS5)**

SMA contains high binder content and extreme care should be taken during the placing process of SMA. Multi-tyred rollers are not used in paving SMA because of the possible working of binder to the surface of the SMA leading to flushing and pick-up. By the same reason, trafficking of newly-placed hot SMA also produces the same effect of flushing the binder to the surface. As such, it is preferable to have the SMA cooled below 40$^o$ before opening to traffic.

**23. Why is anti-skid dressing sometimes applied to pavement surface? (RS2)**

Skid resistance of road pavement plays an importance role when vehicles take a bend or brake. The use of anti-skid dressing increases the friction to reduce the possibility of skidding. Anti-skid dressing refers to the road surface treatment which includes high-friction calcined bauxite as the high Polished Stone Value artificial aggregate, together with a resin (epoxy, polyurethane) instead of bitumen to tie the aggregate to the road surface. It is more expensive but is more durable in difficult sites such as crossing and roundabout. Moreover, it also has side benefits of good acoustic quality.

Anti-skid dressing has normally a different colour to the road surface and used to be red in colour. Anti-skid dressing is commonly used in areas which require higher friction values such as approach to roundabouts and traffic lights.

The ability of the road surface to resist kidding is a combination of the surface texture of the road surface and the Polished Stone Value of the aggregate in the road surface. The higher the Polished Stone Value the higher resistance the aggregate has to polished, and hence the greater the capability the aggregate of retaining its own texture. Surface texture of the road surface is often referred to as macrotexture while texture actually on the surface of the aggregate is known as microtexture.

Bleeding forms a sticky and shiny surface which results in the loss of skid resistance for vehicles in wet weather.

**24. Is the skid resistance of bituminous pavement derived from microtexture of aggregates or texture depth of road surface?**

The skid resistance of road surface is of paramount importance in enhancing road safety. The chance of occurrence of skidding is reduced with an increase of skid resistance.

At low traffic speed (i.e. 50km/hr), the skid resistance is mainly controlled by microtexture of bituminous aggregates. For road speed exceeding 50km/hr, the macrotexture of road surface comes into play which is characterized by texture depth of pavement. Hence, surface texture becomes an important parameter when designing high speed roads and expressway. The typical texture depth for concrete carriageway is 0.7mm. However, to maintain skid resistance provided by 0.7mm texture depth of concrete carriageway, 1.5mm texture depth is required for bituminous pavement in high speed road.

# Level One (Core FAQs)

## ***Part II: Concrete Road***

**1. Why is the slump specified in concrete carriageway comparatively low (30mm) when compared with normal concrete (75mm)? (C1)**

The slump of concrete carriageway is purposely specified to be a relatively low value, i.e. 30mm. For concrete carriageway, traffic loads directly act on concrete pavement surface and therefore the surface strength is detrimental to its future performance. In freshly placed concrete, segregation (may be in the form of bleeding) occurs within the mixture of cement paste and aggregates. The degree of resistance to segregation is related to workability of concrete. If substantial segregation is allowed to take place, then the relatively porous and weak laitance layer will be formed on the carriageway surface and the aggregates will concentrate in the bottom. Hence, concrete which has insignificant bleed possesses a stronger surface layer and is more abrasion resistant. Consequently, a small slump value is specified to increase the wearing resistance of concrete and to achieve a suitable surface texture of concrete pavements.

Moreover, a low-slump concrete facilitates the use of slipforms when constructing the concrete pavement. With concrete of a low slump value, it still remains its compacted shape and is not liable to deform when the paving machines go away. However, if a high slump concrete is used instead, the pavement surface would drop and the edges may deform readily.

**2. How can unreinforced concrete pavement function without mesh reinforcement? (PS1)**

For concrete carriageway, it is normally classified into two types: reinforced and unreinforced concrete pavement. The reinforcement in reinforced carriageway (in the form of mesh) is used for controlling cracking. Then one may query how unreinforced pavement can control cracking without the use of mesh reinforcement. To answer this question, one should pay attention to the features of unreinforced concrete pavement. In accordance with Highways Standard Drawing No. H1109, an approximately 3mm wide groove with a depth of about one-third to one-fourth of slab thickness is designed with a regular spacing (normally 5m). The grooves are designed to be too narrow for stones to fall into when the cracks are open due to concrete contraction. The sectional area in which the groove is located is a plane of weakness and thus this groove acts a potential crack-inducing device in which any potential cracks due to shrinkage and thermal contraction may form. Consequently, the cracks are formed at the base of the groove and thus it would not cause any unpleasant visual appearance on the exposed surface of unreinforced concrete pavement.

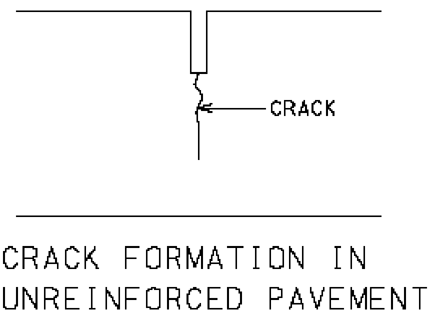



**Fig. Crack formed in unreinforced concrete pavement**

Note: For details of concrete profile barriers, reference is made to HyD Standard Drawing No. H2101A.

### 3. What is the purpose of reinforcement in concrete roads? (PS1)

The main purposes of reinforcement in concrete roads are [21]:

(i) to control the development and pattern of cracks in concrete pavement.
(ii) to reduce the spacing of joints. In general, joints and reinforcement in concrete structures are common design measures to cater for thermal and shrinkage movement. Hence, the inclusion of reinforcement allows the formation of tiny cracks in concrete pavement and this allows wider spacing of joints.

In fact, the amount of reinforcement in concrete slab is not substantial and its contribution to the structural strength of roads is not significant.

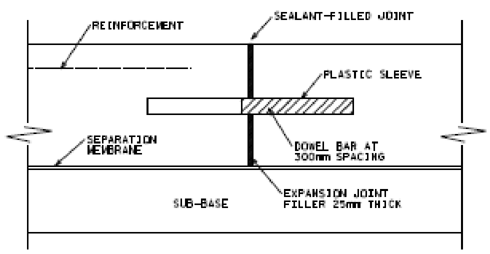



**Fig. Road reinforcement**

**4. In concrete pavement, why is the requirement of 95% by mass of quartz grains are specified in contract?**

In General Specification of Civil Engineering Works (1992 Edition), in Clause 10.09 it states “Fine aggregate for concrete shall be natural river-deposited sand consisting of at least 95% by mass of quartz grains”. The aim of such specification is to control the quality of river sand. As quartz is a durable and hard material, using a high percentage of quartz in aggregates of concrete can enhance the strength and durability of the surface texture of concrete carriageway.

In addition, such specification requires a high percentage of quartz content, thereby reducing the probability of presence of impurities like shell.

**5. Should joints of concrete kerbs be in line with the joints in concrete carriageway? (PS2)**

In normal practice, joints are provided in road kerbs to cater for concrete expansion and contraction. However, the location of joints in kerbs is not arbitrary and they should match with joints in concrete carriageway. Otherwise, it is very likely that cracks may form in concrete kerbs at location of pavement joints (Ministry of Transport (1955)).

**6. Can a sub-base layer serve as a drainage layer to remove water from pavement? (PS3)**

Besides providing load distribution in post-construction stage and working platform in construction stage, sub-base can also serve as a drainage layer to remove water coming from the pavement. For thick layers of road pavement, it is likely that the water leakage from pavement is insignificant and therefore the sub-base layer mainly serves to support the traffic stresses. However, for thin pavement layers, water penetration is quite substantial and therefore sub-base may also act as a drainage layer to remove these water.

**7. If there is a delay of bituminous laying on top of sub-base, should tack coat be applied on the top surface of sub-base? (PS4)**

When there is a delay between bituminous laying of different bituminous layers (i.e. roadbase, base course etc.), a tack coat is applied on top of the bituminous layers because it helps to enhance better bonding between bituminous materials. If there is insufficient bonding between adjacent bituminous layers, they behave as separate independent layers which can hardly resist the traffic loads. When applying the tack coat, it should be sprayed uniformly on the bituminous surface and allowed for sufficient curing. The hot bituminous material laid on top of the coat would soften it, enabling the tack coat to partly fill voids in the bituminous materials. For emulsified asphalt type tack coats, they are normally diluted with water in order to achieve a more uniform application without excessive usage of asphalt. After the subsequent compaction is carried out, the coat would be interlocked with the bituminous materials. On the other hand, care should be taken to ensure that excessive coat would not be laid, otherwise slippage or shear cracks in the bituminous material would occur due to the relative thick layer of the tack coat applied.

However, for sub-base surface, priming coat instead of tack coat may be applied in the event of a delay in laying of bituminous layer on top of the sub-base layer. The function of the primer serves to maintain the existing surface condition for a longer period and it also provides an impermeable surface to prevent ingress of water or water loss by evaporation. Moreover, it fills the surface voids and protects the sub-base from adverse weather conditions. In addition, it also helps to promote adhesion between adjacent road layers and to harden the surface.

**8. For rigid pavement, what are the advantages of using lean concrete sub-base instead of traditional granular sub-base? (PS3 & PS7)**

There are several shortcomings of using granular sub-base in concrete carriageway:

(i) Since sub-base is permeable, water can seep through sub-base and soil particles will be pumped out through contraction/expansion joints when subject to traffic load. Consequently, voids are formed underneath the pavement structure and the concrete pavement may crack under severe traffic loading.
(ii) Lean concrete increases the strength and renders the roads capable of carrying heavy traffic loads (David Croney and Paul Croney (1992)).
(iii) Due to workmanship problem, it may have uneven distribution of sub-base and this results in cracking of concrete carriageway when subject to severe traffic loading.

**9. Does sub-base of concrete carriageway provide strength support? (PS3)**

Basically, sub-base for a concrete carriageway is provided for the following reasons [55]:

(i) It provides a smooth and even surface between the subgrade and concrete slab. This avoids the problem of uneven frictional stresses arising from the uneven interface under thermal and shrinkage movement. It also improves the uniformity of support provided to concrete slab to enhance even distribution of wheel load to the subgrade.
(ii) For heavily trafficked carriageways with frequent occurrence of a high water table, it serves to prevent the occurrence of mud pumping on clayey and silty subgrade. The loss of these clayey soils through carriageway joints such as contraction and expansion joints will cause structural failure of concrete slab under heavy traffic load.

The stiffness of concrete slab accounts for the strength of rigid road structure. It is normally uneconomical to employ sub-base as part of the strength provider because a much thicker layer of sub-base has to be adopted to reduce the thickness of concrete slab by a small amount. Hence, it is more cost-effective to increase the depth of concrete slab rather than to enhance foundation strength in order to achieve a higher load-carrying capacity of the concrete pavement.

**10. What are the differences between capping layer and sub-base? (PS5)**

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.

**11. Why should a uniform and regular sub-base surface be provided in concrete pavement? (PS3)**

The surfaces of sub-base material for concrete carriageway should be constructed in a regular manner because of the following reasons [21]:

(i) One of the main functions of sub-base in concrete pavement is to provide a smooth and even interface between concrete slab and subgrade so that a uniform support is established. A regular surface of sub-base assists in reducing the frictional and interlocking forces between concrete slab and sub-base and allowing easier temperature and shrinkage movement.
(ii) A uniform sub-base surface is essential in the construction of concrete slab of uniform thickness adopted in design. It saves the higher cost of concrete to make up the required level.

**12. What is the function of waterproof (or separation) membrane for concrete carriageway? (PS6)**

A layer of waterproof (or separation) membrane is normally placed between sub-base and concrete slab for the following reasons [21]:

(i) It prevents the loss of water from cement paste which affects the strength of concrete slab.
(ii) It enhances the movement of concrete slab relative to sub-base layer and reduces the frictional forces developed at their interface.
(iii) It avoids the possibility of active aggressive agents from soil water being attached to the concrete slab.
(iv) It prevents the intermixing of freshly placed concrete with loose materials on the surface of sub-base.

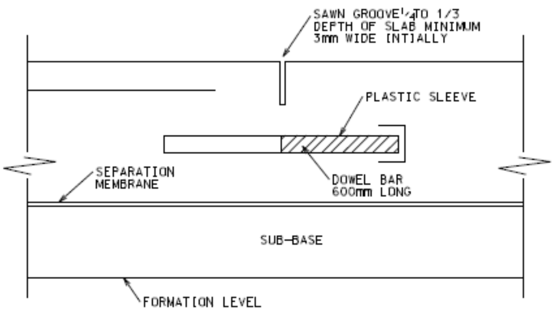



**Fig. Location of separation membrane in concrete carriageway**

**13. What is the purpose of using capping layers in pavement construction? (PS5)**

When the California Bearing Ratio of subgrade is checked to be below a certain percentage (e.g. 5%), a capping layer is normally provided to reduce the effect of weak subgrade on the structural performance of the road. It also provides a working platform for sub-base to be constructed on top in wet weather condition because the compaction of wet subgrade is difficult on site. The effect of interruption by wet weather can be reduced significantly and the progress of construction works would not be hindered. Most importantly, the cost of capping layers is low because the material can be readily obtained locally.

**14. Should lean concrete base and concrete slab be bonded together? (PS7)**

There are two schools of thought regarding the suitability of bonding between lean concrete base and concrete slab. The first one considers it undesirable to bond them together to minimize reflection cracking in the slab. In essence, bonding would cause cracks in lean concrete base to reflect through the concrete slab which may lead to premature cracking. Hence, bond breaking is achieved by applying wax curing compounds on lean concrete bases.

The other school of thought is to promote bonding between lean concrete base and concrete slab by treating the base with a rough texture to promote bonding to the slab. They consider that debonding lessens the lean concrete base's structural contribution to the pavement and increases stresses due to curling and warping.

**15. For concrete carriageway, which type of subbase is better, granular subbase or lean concrete (cement-bound material)? (PS3 & PS7)**

Subbase serves the following functions in a concrete pavement:

(i) Aid in spreading load of road traffic.
(ii) Prevent pumping of fines from subgrade.
(iii) Protect the subgrade from frost.
(iv) Improve drainage of the pavement structure so that the pavement structure is mostly dry with minimum moisture.

Cement-treated bases are easy to construct and provide a stronger and more erosion-resistant platform for the concrete slab when compared with granular subbase. They are normally used for medium to high traffic loads. However, tensile stresses arising from temperature curling would be significantly increased by the presence of lean concrete subbase as compared to granular subbase. Moreover, there is an increase in frictional resistance between concrete slab and subbase.

Granular subbase is susceptible to erosion and it retains moisture for long periods. They are normally used only for roads with low to medium traffic loads.

**16. Why does “pumping” sometimes occur at joints in concrete carriageway?**

Pumping at joints in concrete carriageway occurs in the presence of the following factors:

(i) Fine-grained subgrade;
(ii) Seepage of water into subgrade due to improper or inadequate drainage design;
(iii) The presence of heavy vehicular loads.

It involves the pumping out of water-borne particles of the subgrade owing to the deflections at the end of concrete slab. The first mechanism of pumping involves the softening of subgrade by water and the reduction in bearing capacity. It causes a larger instantaneous deflection at the slab ends under heavy traffic loads. During deflection, water containing fine soil particles is pumped out at the joints. Consequently, voids are formed in subgrade region and the void size grows by repeating the above sequence [21].

**17. What are the potential problems in corners for concrete pavement? (PS8)**

Consider a panel of concrete slab without any load transferring devices at it edges. When the concrete panel is subjected to traffic loads, the maximum stress induced in the concrete panel is at its four corners. Other than panel corners, the next significant stress induced in concrete slab is its four edges.

To avoid the structural failure of concrete pavement, one can locally increase the thickness of corners and edges to reduce the induced stresses. However, such local thickenings also increase temperature stresses. Moreover, the construction of non-uniform concrete pavement is not convenient from practical point of view. The other way out is to use load transfer devices like dowel bars at the edges of concrete panels. However, in situation where the designed thickness of concrete pavement is small which renders the provision of dowel bars not practical, special design of corner reinforcement has to be considered [21].

**18. Should engineers design acute angle for concrete pavement? (PS8)**

The stress induced in acute angle corners of concrete pavement is far much higher than that in right-angle corners of the pavement. For instance, concrete pavement corner of acute angle of $70^o$ induces stresses about 50% more than the stress induced by an angle of $90^o$. As a result, corners of concrete pavement should not be designed with acute angles to avoid corner cracking. If it is necessary to adopt acute angles for concrete pavement, special reinforcement has to be provided to strengthen these corners [55].

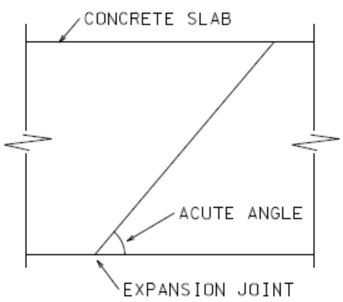



**Fig. Acute angle of concrete pavement**

**19. There is an optimum time to saw contraction joints in new concrete pavement. Why? (C2)**

There is an optimum time to saw contraction joints in new concrete pavement. Sawing cannot be carried out too early because the saw blade would break out particles from pavement and this results in the formation of jagged and rough edges. Such raveling is undesirable as it would impair the appearance and the ability to seal the joint properly.

Sawing cannot be implemented too late. When the volume of concrete is reduced significantly owing to drying shrinkage or thermal contraction, it induces tensile stress owing to the restraint of such reduction. When the tensile stresses exceed the tensile strength of concrete, cracking would result and form in other locations instead of the planned location of contraction joints.

**20. For box-out of concrete carriageway, should square box-out or round box-out be adopted? (C3)**

Isolation joints are introduced where the pavement contains manholes and other structures. The joints allow independent movement of the pavement and roadside structures without any connection which otherwise could result in damage.

Square box-outs are commonly used in pavement and it suffers from the demerit that cracks are observed to appear at the corners of the box-out. Hence, to prevent crack formation at corners, the use of rounded box-outs could eliminate such occurrence. In fact, some form of reinforcement could be introduced to reduce the crack formation in square box-outs. For instance, fillets or reinforcing bars and fabric could be placed at interior corners of square box-outs to hold the cracks together should cracks grow.

**21. What is the significance of ten percent fines value in testing sub-base material?**

Ten percent fines value is a measure of the resistance of aggregate crushing subjected to loading and it is applicable to both weak and strong aggregate. Fine aggregates are defined as those passing 2.36mm sieve. The test aims at looking for the forces required to produce 10% of fine values (i.e. weight of fines aggregates/weight of all aggregates = 10%). This test is very similar to Aggregate Crushing Test in which a standard force 400kN is applied and fines material expressed as a percentage of the original mass is the aggregate crushing value.

Granular sub-base is subjected to repeated loadings from truck types. The stress level at the contact points of aggregate particles is quite high. The sub-base in pavement is a structural layer used for distribution of traffic loads into larger area. As such, it is of paramount importance that the sub-base material should itself not be disintegrated under severe traffic loads. Ten percent fines value can be used to reveal the aggregate properties when subjected to mechanical degradation.

### 22. What is difference in purpose for conducting surface regularity test and sand patch test (test on texture depth)? (T2)

The purpose of surface regularity test is to measure the riding quality of pavements and the same requirement and standard is applied to both concrete and bituminous carriageway. On the other hand, sand patch test is used for checking the skid resistance of road pavements. Moreover, it is related to traffic noise because the intensity of noise generated from road traffic is related to texture depth of carriageway.

### 23. 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.

## Level One (Core FAQs)

### ***Part III: Paving Blocks***

**1. How do concrete paving blocks take up loads? (S1)**

The paving for concrete blocks consists of closely packed paving blocks in pre-determined patterns and the tiny joint spaces between individual blocks are filled with sand. The presence of sand avoids the displacement of a single block unit from the remaining blocks. Moreover, the horizontal interlocking provided by the arrangement of paving blocks in special patterns (e.g. herringbone pattern) prevents any single block from moving relative to one another. For instance, vertical loads acting directly on one concrete paving block are not only resisted by the block itself, but also by the blocks adjacent to it [59].

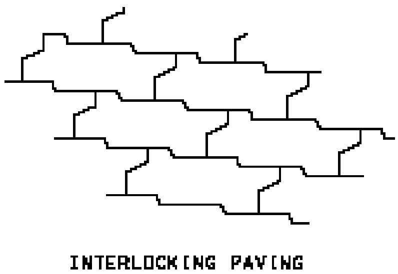



**Fig. Paving blocks**

**2. What is the load transfer mechanism for paving blocks in pedestrian footway? (S1)**

In Hong Kong, footway is normally designed with paving slabs/interlocking blocks instead of concrete because the extensive use of concrete in pavement is boring. Moreover, poor appearance will result in concrete pavement due to frequently trench openings for utility work. In addition, owing to the prolonged setting and curing time of concrete, the use of concrete pavement inevitably generates disturbance to the general public.

The pedestrian loads are taken up through the following ways:

(i) Load carrying capacity of paving slabs/interlocking blocks are derived from their individual strength;
(ii) Pedestrian loads are also supported by interlocking forces provided by friction transfer through the sand in vertical joints (K. K. Tang &

Robert P. Cooper (1986)).

**3. What is the function of a sand layer underlying paving slab/interlocking blocks? (S2)**

Normally after the laying of sub-base layer of the paving slab/interlocking blocks, a 30mm thick sand bedding is screeded and tamped over the pavement area. Then paving slabs are laid horizontally with joints of 2-3mm wide and are laid in uphill direction. After completing the laying of paving slabs, sand used for filling joints is spread over the surface of the units and brushed into the joints such that all joints are completely filled. The paving slabs are then bedded into final position by using plate vibrators.

The sand layer serves the same purpose of normal blinding layer under concrete structure:

(i) Provide a level and flat surface for the paving slab/interlocking blocks to lay on;
(ii) Protect the foundation (i.e. underground subgrade and sub-base layer) against adverse outside conditions (e.g. bad weather) during construction of the laying work of paving slab/interlocking blocks.

**4. Can the sand bedding be omitted in paving block pavement? (S2)**

In the paving block pavement, it normally consists of the following main elements: sub-base, sand bedding and paving blocks. Sub-base is the main structural element to take up traffic load and spread it into larger area so that the traffic stress is small enough for subgrade to sustain. Sand bedding is used for providing correct line and level for paving blocks to lie on. To achieve this, screeding of bedding layer is implemented so that the paving blocks could be laid directly on it without the need of further leveling. The large range of particle sizes associated with sub-base renders it unsuitable to provide a uniform surface with correct level for paving blocks to lie on.

**5. Should sand layer or cement sand be used as bedding of precast concrete paving units? (S2)**

Cement sand is a mixture of cement and sand and it acts as a cohesive mass once mixed. Normally, a 20mm to 30mm sand layer is laid underneath precast paving block units. However, in locations of steep gradients where it stands a high possibility that rain runoff will wash out infilling sand and sand layers, cement sand should be sued instead. Similarly, when high pressure jetting is anticipated to be employed frequently in routine maintenance, sand layers beneath precast paving block units is not preferable owing to the reason of potential washing out of sand.

### 6. Why should edge courses of paving blocks sit on concrete bedding? (S3)

In pavement made up of paving blocks, the edge courses are normally designed to sit on concrete bedding and haunching. The reason of such provision is to prevent lateral movement of paving blocks when subjected to traffic loads. In essence, the edges formed by edge courses tie the body of the paving blocks as a single unit and they have to rely on concrete bedding and haunching to resist the sideway forces generated by moving traffic.

The concrete bed serves to keep the edge courses in position for surface level and it provides the dead loads in the retaining structure. On the other hand, concrete haunching also holds the edge courses in position from the point of view of lateral movement.

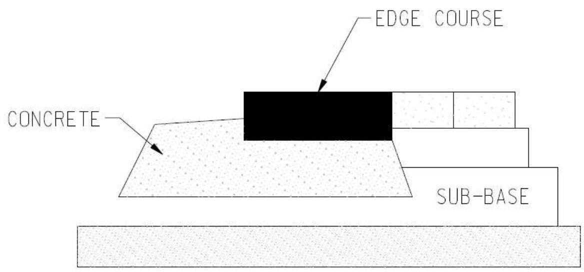



**Fig. Edge courses in paving blocks**

### 7. What is the purpose of edge courses in paving blocks pavement? (S3)

The principal function of edge courses is to form a retaining structure with concrete bedding and haunching to resist sideway movement arising from traffic loads. Other functions of edge courses include:

(i) It serves as level guides when carrying out screeding of sand bedding.

(ii) It aids in the cutting of paving blocks near edge courses because paving blocks could lie over edge courses and the positions of cutting could be marked accurately without any difficulty. However, if a wall instead of edge courses is located adjacent to the pavement, it is practically difficult to position the paving block over the gap because of the physical hindrance of the wall.
(iii) It could be designed as surface drainage channel to convey stormwater to a discharge point.
(iv) It acts as a frame which defines the shape of pavement.

**8. For compaction of paving blocks, should the jointing sand be applied before or after the compaction process? (S4)**

The common practice is to apply jointing sand at the first place followed by subsequent compaction. However, there are several potential problems associated with this method. Firstly, the presence of sand provides support to paving blocks leading to inadequate compaction. Secondly, the excess jointing sand may be crushed during compaction and leaves stains on the surface of paving blocks. Thirdly, damaged paving blocks appear to be difficult for removal owing to frictional grip by jointing sand.

The alternative method is to carry out compaction firstly and then followed by application of joint sand and then re-compaction is carried out again. This method eliminates all the shortcomings of the first method described above. However, it suffers from the demerit of two passes of compaction instead of a single stage of compaction is required. Moreover, the compaction operation tends to be noisier because of the absence of the infilling sand which helps to reduce noise level. The direct contact between individual paving blocks is more vulnerable to spalling during compaction.

**9. Can all utility detectors detect the depth of utilities?**

In Hong Kong, underground utility detectors are normally divided into two types: electromagnetic detector and ground penetrating radar (GPR).

For electromagnetic detector, it can detect the signals emitted by metallic utilities themselves by passive mode. While in active mode, the detector has to pick up the signals through a transmitter and sonda connected directly to the non-metallic utility. Both the alignment and depth can be found in active mode while only alignment can be found in passive mode. The electromagnetic detector available in market can detect utilities up to depth of 3m.

For ground penetrating radar, it sends radio waves into the ground and receives signals from reflections from utilities. It has the advantage of locating both the depth and alignment of utilities. More importantly, it can detect both metallic and non-metallic utilities. However, it suffers from the disadvantage that it is quite expensive and interpretation of data is not simple. Reference is made to LD, DSD (2000).

## Level Two (Advanced FAQs)

### ***Part I: Road Joints***

**1. Which joint sealant is better, acrylic, polysulfide, polyurethane or silicone? (RJ1)**

There are four generic types of joint sealant with high performance. Their properties are highlighted in the following table:

| Acrylic | Polysulfide | Polyurethane | Silicone |
|---|---|---|---|
| • Accommodate 12% movement.<br>• Exhibit shrinkage upon curing<br>• Solvent-based. | • Poor recovery in high cyclic movements<br>• Exhibit excellent chemical resistance<br>• Good performance in submerged conditions. | • Accommodate 50% movement.<br>• Excellent bonding, can be used without primer<br>• Good UV resistance | • Accommodate 50% movement.<br>• Excellent low temperature movement capability<br>• Excellent UV and heat stability |

**2. Can joint sealant provide a perfect watertight seal in joints? (RJ1)**

The two main principal functions of joint sealant are to minimize the entry of surface water and prevent the ingress of incompressible material from entering the joint. Other minor function of joint sealant is to reduce the possibility of corrosion of dowel bar by the entrance of de-icing chemicals.

Water entry into joints is undesirable because it leads to the softening of subgrade and pumping of subgrade fines under heavy traffic. However, it is impractical to maintain a completely watertight pavement structure. In fact, vacuum tests show that no sealants could provide 100% watertight seal. The current philosophy to combat water ingress into joints is only to minimize but not to completely prevent water from entering the pavement structure. Instead, a permeable subbase is designed to remove water from the pavement.

**3. In expansion joints why are plastic sleeve normally used in dowel bars instead of debonding agent? (RJ2)**

The purpose of plastic sleeve or debonding agent around dowels bars in expansion joints is to minimize the frictional resistance between the bar and its surrounding concrete. This is the reason why plain round bars are usually used instead of deformed bars which provide mechanical interlock with concrete and hence it hinders the free movement of the dowel bar.

Both bituman-based paint (debonding agent) and plastic sleeve could serve the purpose of reducing friction between dowel bar and surrounding concrete. From practical point of view, the use of plastic sleeve (e.g. PVC dowel sleeve) around dowel bars can well prepared off-site and manufactured well in advance, thus saving the time of construction.

**4. For unreinforced concrete carriageway, what is the sequence of closing and opening of expansion joints and contraction joints?**

Let's take an example to illustrate the sequence of closing and opening of joints (Ministry of Transport (1955)). For instance, an unreinforced concrete carriageway is constructed in winter. When temperature rises in the following summer, the section between expansion joints will expand as a whole single element resulting in the closure of expansion joints. This section of concrete pavement will move outwards from the mid-point between the expansion joints. In the next winter, each bay (i.e. concrete pavement between adjacent contraction joints) of concrete contracts about the midpoint of its length with opening of contraction joints.

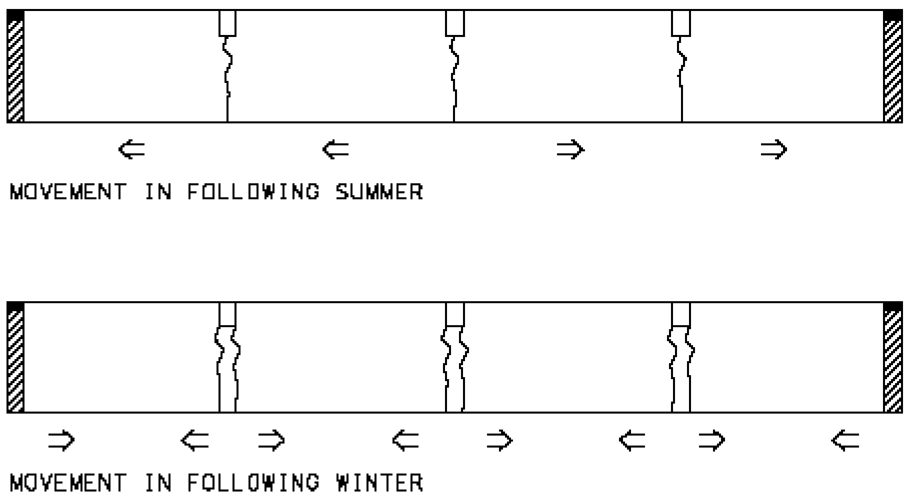



**Fig. Movement of joints in summer and winter**

**5. If the construction of concrete carriageway is carried out in summer, can expansion joints be omitted?**

If the construction of concrete carriageway is carried out in summer, expansion joints may not be necessary as suggested by Arthur Wignall, Peter S. Kendrick and Roy Ancil. Expansion of concrete carriageway is mainly due to seasonal changes with an increase in temperature from that during construction to the ambient temperature (i.e. the temperature in summer). However, if the construction of concrete carriageway takes place in summer, the concrete carriageway will undergo contraction in the following winter, thus the space available in contraction joints can accommodate the future expansion in the next summer.

**6. What is the function of longitudinal joints in concrete road pavements? (RJ3)**

A longitudinal joint consists of a tie bar placed at the mid-depth of a concrete pavement and it is not intended for joint lateral movement. Then one may doubt the reasons of placing longitudinal joints in concrete pavements. In fact, longitudinal joints are normally designed at a regular spacing e.g. 4.5m to accommodate the effect of differential settlement of pavement foundation. When uneven settlement occurs, the tie bars in longitudinal joints perform as hinges (Ministry of Transport (1955)) which allow for the settlement of concrete carriageway. Moreover, it also serves to cater for the effect of warping of concrete due to moisture and temperature gradients by permission of a small amount of angular movement to occur so that stresses induced by restrained warping can be avoided.

Dowel bars are provided in longitudinal joints for the following reasons:
(i) In case of the occurrence of uneven settlement between adjacent panels, it helps to maintain a level surface by transfer of loads through dowel bars.
(ii) Keep the longitudinal joints close.

**7. Why are contraction joints in concrete pavement normally designed to be unsealed? (RJ4)**

For unreinforced concrete pavement, the contraction joint is an approximately 3mm wide groove with a depth of about one-third to one-fourth of slab thickness and a regular spacing of normally 5m. The grooves are designed such that they are too narrow for stones to fall into when the cracks are open due to the contraction of concrete. The groove location is a plane of weakness and the groove acts as a potential crack-inducing device where any potential cracks due to shrinkage and thermal contraction may form will be confined to the base of the groove. It will not cause any unpleasant visual appearance on the exposed surface of unreinforced concrete pavement.

The above-mentioned contraction joints can be designed as unsealed. These grooves are very narrow so that stones can hardly get into these grooves even when the joint undergoes contraction. The fine particles or grit entering into the groove are likely to be sucked out by the passing vehicles. The joints can be self-cleansing and it may not be necessary to seal the joints for fear of attracting the accumulation of rubbish and dirt [55].

**8. What is the difference between tie bars and dowel bars in concrete carriageway? (RJ5)**

Tie bars are deformed rebars or connectors used for holding faces of rigid slabs in contact to maintain aggregate interlock. Tie bars are not load transferring device. For instance, tie bars are used in longitudinal joints in concrete pavement.

Dowel bars are smooth round bars which mainly serve as load transfer device across concrete joints. They are placed across transverse joints of concrete pavement to allow movement to take place. Where movement is purposely designed for longitudinal joints, dowel bars can be adopted.

**9. In concrete pavement, keyway joint are sometimes adopted in longitudinal joint. Why? (RJ6)**

Longitudinal joints are installed in concrete pavement to prevent differential settlement between adjacent concrete panels. Moreover, it serves to control cracking from stresses caused by volumetric changes of concrete owing to moisture and thermal gradients. In essence, the joint contains tie bars to enhance efficient load transfer between adjacent concrete panels.

Sometimes keyway joint are designed in longitudinal joint to improve the performance of the joint. Though the installation of keyway joint does not appear to increase the load transfer efficiency of longitudinal joint, it proves to help reduce the deflection of concrete pavement. Keyways are not recommended for thin slab (less than 250mm thick) because of the difficulty in construction. Moreover, keyways are prone to failure in thin concrete slabs where they are too large or too close to slab surface.

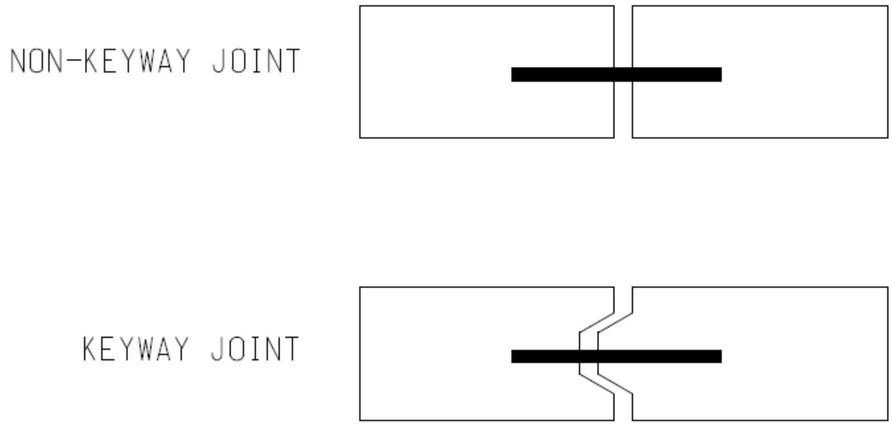



**Fig. Non-keyway joint and keyway joint**

**10. For a concrete pavement constructed over a box culvert, why is it important to match the location of transverse joints with boundary of box culvert?**

When a concrete pavement is constructed over a box culvert, it is important to match the location of transverse joints with boundary of box culvert. Otherwise, it is likely that full-depth transverse cracks would develop on the pavement slabs just above the location of boundary of box culvert. In case a layer of granular sub-base is introduced to place under the pavement slabs, the sub-base layer serves as crack-arresting layer and the possibility of development of transverse cracks in concrete pavement is reduced.

# Level Two (Advanced FAQs)

## *Part II: Pavement Design*

### 1. What is the principle of asphalt mix design? (AMD1)

The main objective of asphalt mix design is to achieve a mix with economical blending of aggregates with asphalt to achieve the following [61]:

(i) workability to facilitate easy placement of bituminous materials without experiencing segregation;
(ii) sufficient stability so that under traffic loads the pavement will not undergo distortion and displacement;
(iii) durability by having sufficient asphalt;
(iv) sufficient air voids

In asphalt mix design, high durability is usually obtained at the expense of low stability. Hence, a balance has to be stricken between the durability and stability requirements.

### 2. What is Marshall Mix Design for bituminous materials? (AMD2)

The Marshall Mix Design method was originally developed by Bruce Marshall of the Mississippi Highway Department in 1939. The main idea of the Marshall Mix Design method involves the selection of the asphalt binder content with a suitable density which satisfies minimum stability and range of flow values.

The Marshall Mix Design method consists mainly of the following steps:

(i) Determination of physical properties, size and gradation of aggregates.
(ii) Selection of types of asphalt binder.
(iii) Prepare initial samples, each with different asphalt binder content. For example, three samples are made each at 4.5, 5.0, 5.5, 6.0 and 6.5 percent asphalt by dry weight for a total of 15 samples. There should be at least two samples above and two below the estimated optimum asphalt content.
(iv) Plot the following graphs:
    (a) Asphalt binder content vs. density
    (b) Asphalt binder content vs. Marshall stability

(c) Asphalt binder content vs. flow
(d) Asphalt binder content vs. air voids
(e) Asphalt binder content vs. voids in mineral aggregates
(f) Asphalt binder content vs voids filled with asphalt

(v) Determine the asphalt binder content which corresponds to the air void content of 4 percent

(vi) Determine properties at this optimum asphalt binder content by reference with the graphs. Compare each of these values against design requirements and if all comply with design requirements, then the selected optimum asphalt binder content is acceptable. Otherwise, the mixture should be redesigned.

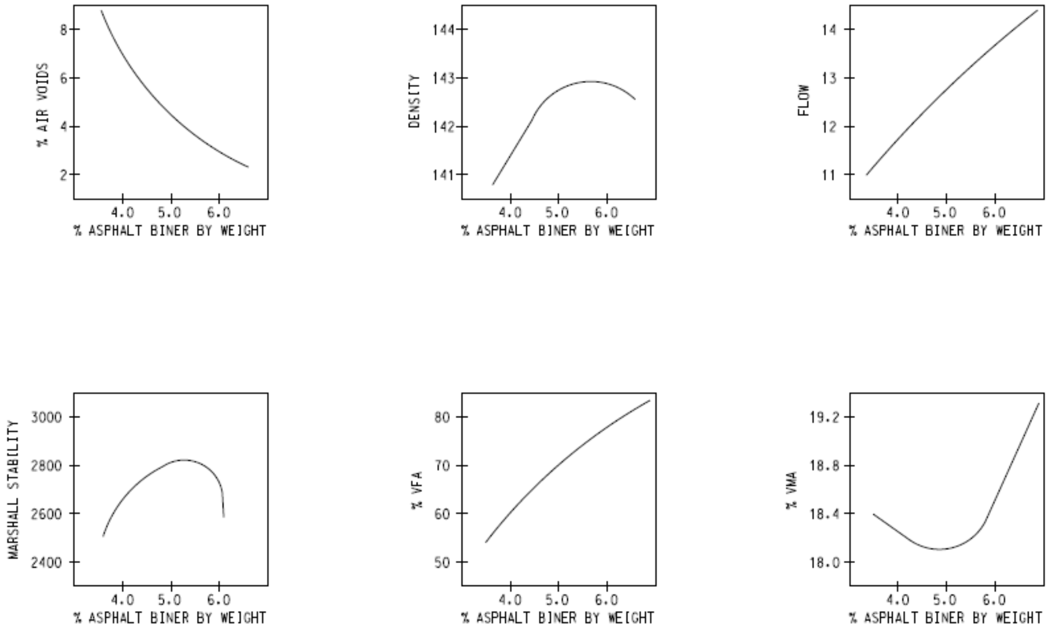



**Fig. Design graphs for Marshall Mix Design**

## 3. What is the importance of Marshall stability and flow test? (AMD3)

Marshall stability measures the maximum load sustained by the bituminous material at a loading rate of 50.8 mm/minute. The test load is increased until it reaches a maximum. Beyond that, when the load just starts to decrease, the loading is ended and the maximum load (i.e. Marshall stability) is recorded. During the loading test, dial gauge is attached which measures the specimen's plastic flow owing to the applied load. The flow value refers to the vertical deformation when the maximum load is reached.

Marshall stability is related to the resistance of bituminous materials to distortion, displacement, rutting and shearing stresses. The stability is derived mainly from internal friction and cohesion. Cohesion is the binding force of binder material while internal friction is the interlocking and frictional resistance of aggregates. As bituminous pavement is subjected to severe traffic loads from time to time, it is necessary to adopt bituminous material with good stability and flow.

**4. Should large amount or small amount of air voids be designed in bituminous pavement? (AMD4)**

If the presence of air voids is too high, it leads to an increase of permeability of bituminous pavement. This allows the frequent circulation of air and water within the pavement structure and results in premature hardening and weathering of asphalt. Therefore, too high an air void content poses detrimental effect to the durability of the bituminous pavement.

If the presence of air voids is too low, flushing, bleeding and loss of stability may result under the effect of prolonged traffic loads because of the rearrangement of particles by compaction. Aggregates may become degraded by traffic loads leading to instability and flushing for such a low air void content. The air void space can be increased by adding more course or fine aggregates to the asphalt mix. Alternatively, if asphalt content is above normal level, it can be reduced to increase air voids [61].

**5. In General Specification for Civil Engineering Works (1992 Edition), the design of roadbase material is based on recipe approach. Why? (AMD5)**

The design of roadbase material is based on recipe approach (David Croney and Paul Croney (1992)) because Hong Kong government follows the traditional British practice by adopting recipe design in which the aggregate grading envelope, the quantity and grade of bitumen are specified in the bituminous mix. This recipe of bituminous mix is derived based on past experience and good workmanship during construction. In fact, many countries nowadays adopt special design mix of roadbase which proves to produce satisfactory bituminous mixes to suit different site and design conditions.

In fact, recipe specification of bituminous materials does suffer from several drawbacks. Firstly, the conditions of traffic and climate of newly constructed bituminous road may differ from the conditions on which the recipe design is based. In case adjustment has to be made to the recipe design, it is very difficult to determine and assess the modifications required. Secondly, it poses problem to site engineers to assess the effects of minor non-compliance if recipe specification is adopted. Finally, the recipe mix may not be the most economical design which is dependent on site conditions.

**6. Should high-yield steel or mild steel be designed as road reinforcement? (SR1)**

High yield steel is the preferred material for the reinforcement of concrete carriageway because of the following reasons [55]:

(i) The principal function of steel reinforcement in concrete pavement is to control cracking. If mild steel is adopted for reinforcement, upon initiation of crack formation mild steel becomes overstressed and is prone to yielding. High yield steel offers resistance to crack growth. The above situation is commonly encountered where there is abnormal traffic loads on concrete carriageway exceeding the design limit.
(ii) High-yield steel is less prone to deformation and bending during routine handling operation.
(iii) In the current market, steel mesh reinforcement is normally of high-yield steel type and the use of mild steel as road reinforcement requires the placing of special orders to the suppliers.

**7. In which direction should the main weight of reinforcement be placed in concrete pavement? (SR2)**

The reinforcement of concrete pavement is usually in the form of long mesh type. A road usually has length is generally much longer than its width and therefore cracking in the transverse direction has to be catered for in design. Reinforcement is required in the longitudinal direction to limit transverse cracking while transverse steel acts to provide rigidity to support the mesh fabrics. For long mesh in concrete slab, the main weight of reinforcement should be placed in the critical direction (i.e. longitudinal direction) to control cracking. However, if the concrete road is quite wide, certain reinforcement has to be placed in the transverse direction in this case to control longitudinal cracking [55].

**8. What are the design considerations for dowel bars in joints of concrete carriageway?**

The behaviour of dowel bars resembles that of piles in soils subject to lateral loads. Failure of the joint occurs by yielding of steel under bending action or by crushing of concrete due to bearing stresses.

In general, the spacing of dowel bars should not be too close which may pose problems during construction. However, it should be not too wide to allow the occurrence of bending between the dowel bars. On the other hand, regarding the length of dowel bars, it should not be too long because the induced stress at the end of long dowel bars is insignificant and is not effective in transferring loads between adjacent concrete panels. However, if the dowel bars are too short, the stress at the face of joint is increased resulting in concrete crushing. Reference is made to Ministry of Transport (1955).

**9. Is California Bearing Ratio method suitable for pavement design? (RF1)**

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.

**10. Can subsoil drains remove all moisture to protect road formation? (RF2)**

Water control is essential to enhance subgrade to possess good bearing value and strength. Water movement in soils takes place by the action of gravity and capillary actions. Subsoil drains are commonly placed at least 1m below finished subgrade to maintain a lower groundwater table.

A properly constructed subsoil drain could effectively lower groundwater table and control the capillary rise. However, they may not be able to eradicate completely the upward movement of moisture in soils. Therefore, a sand layer or granular sub-base could be placed on sub-grade to remove and intercept the moisture once it starts to accumulate.

**11. Which of the following cause much damage to bituminous pavement?**

**(a) Low usage by heavy vehicles or frequent usage of light vehicles;**

**(b) Low speed traffic or high speed traffic**

The relationship between axle weight and the associated pavement damage is not linear but exponential. The pavement damage caused by one passage of a fully loaded tractor-semi trailer (80kN) is more than 3,000 passages of private cars (9KN). Hence, heavy trucks and buses are responsible for a majority of pavement damage.

Slow-moving traffic imposes greater damage than fast-moving traffic. Past studies showed that when the speed is increased from 2km/hr to 24km/hr, the stress and pavement deflection is reduced by 40%.

**12. In the backcalculation of moduli in Falling Weight Deflectometer, why are non-unique solutions generated? (RS2)**

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.

**13. Which method of measuring road stiffness is better, Benkelman Beam Test or Falling Weight Deflectometer Test? (RS1 & RS2)**

Pavement surface deflection measurement is the principal means of evaluating a flexible pavement structure because the magnitude and shape of pavement deflection is a function of traffic, pavement structure, temperature and moisture affecting the pavement structure. Deflection measurements can be used in back calculation method to determine the stiffness of pavement structural layers.

The Benkelman Beam measures the *static deflection*s and it is operated on the basis of lever arm principle. Measurement is made by placing the tip of the beam between the dual tires and measuring the pavement surface rebound as the truck is moved away. The test is of low cost but it is time consuming and labour intensive in carrying out the test.

In Falling Weight Deflectometer Test, the falling weight deflectometer is mounted in a vehicle. The sensors are lowered to the pavement surface and the weight is dropped. The test measures the *impact load response* of flexible pavement. It has the potential advantages that it is quick to perform and the impact load can be readily changed. Moreover, the impact action of falling weight appears to be more accurately representing the transient loading of traffic.

# Level Two (Advanced FAQs)

## ***Part III: Road Furniture***

### 1. Why are concrete profile barriers designed with curved surface profiles? (CBP1)

Safety fencings are designed to contain vehicles in the carriageway in which they are traveling and prevent them from rebounding into the road and causing hazards. For normal fencing design, when vehicles crash into safety fencings, it will give way so as to absorb as much energy as possible, thus reducing the impact forces on the vehicles. Moreover, it serves to realign the vehicles along the carriageway when vehicles hit on them. However, for concrete profile barriers they are not designed to absorb energy during vehicle crashing, but to hold the vehicles hitting on them. In this connection, concrete profile barriers are designed with curved profiles so that vehicles can mount and go up partly on them, and yet they will not cause overturning of vehicles. Reference is made to Arthur Wignall, Peter S. Kendrick and Roy Ancil.

For shallow-angle crashing of cars, they would climb on the lower slope face of concrete profile barriers. On the other hand, when a car hits at a large angle to the barrier, the bumper collides with the upper sloping face of concrete profile barrier and the car rides upwards. This provides the uplift of the car whose wheels move up the lower sloping face of the barrier. It is not intended to lift the car too high, otherwise it may result in rolling. Since the friction between the wheels and barriers provide extra lifting forces, it is undesirable to design rough finish on these faces. In essence, the kinetic energy of the car during collision is transformed to potential energy during its lifting up on profile barrier and finally converted back to kinetic energy when the car returns to the road.

### 2. What are the advantages of having corrugation in crash barriers? (CBP1)

The layout of corrugated beam barriers is that the beams are corrugated in the longitudinal direction so that it provides higher lateral stiffness with a thinner material. Moreover, the distance of beams posts and crashing vehicles are considerably increased.

In case the beam barriers are tensioned, it is intended to create a stiff beam erected on relatively weak posts. During vehicle collision, the posts would be separated from the beams and there would be lesser deceleration experienced by the vehicles [48].

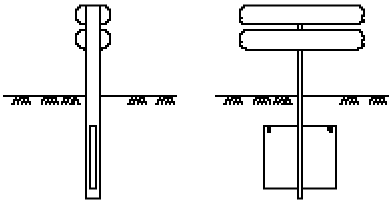


**Fig. Beam barrier.**

**3. Why is concrete crash barriers normally used in central divider? (CPB2)**

Concrete crash barriers are not considered as the best barrier design because of the following reasons when compared with flexible barriers:

(i) Concrete barriers possess rough surface which, when impacted by moving vehicles, tend to cause considerable damage to the vehicles.
(ii) Since concrete is a rigid material and the deceleration of collided vehicles is comparatively large when compared with flexible barriers.

However, concrete crash barriers have particular application in locations where the deflection of barriers is not allowed. For instance, in the central divider of a carriageway, if flexible barriers are adopted and vehicles crush into the barriers, the deformation resulting from the hitting of vehicles would result in an intrusion to the adjacent carriageway. This is undesirable because this may trigger further collisions in the adjacent carriageway and hence rigid barriers like concrete crash barriers should be adopted in this scenario.

**4. What is the difference between safety fence and safety barrier? (BT1)**

A safety fence is intended to absorb some energy caused by hitting vehicles and to realign the vehicles to move parallel to the safety fence.

A safety barrier is intended to provide containment instead of energy absorption upon hit by vehicles. Hence, it is anticipated to have little deflection and deformation only. After crashing, it serves to re-direct the vehicles along the line of barrier.

**5. What are the differences between tensioned corrugated beam and untensioned corrugated beam? (BT2)**

Tensioned corrugated beam is mainly used in high speed road while untensioned corrugated beam is mainly used in low speed road.

Tensioned corrugated beam is designed mainly for use on central reservation. When a vehicle crashes into tensioned corrugated beam, the beam remains in tension while the post gives way to allow for deformation. Tensioned corrugated beam absorbs impact energy by deflecting as a whole and helps the vehicle decelerate, and at the same time guide it back towards the carriageway in a gradual and controlled manner. Tensioned corrugated beam is normally not used on curves with radius less than 120m.

Untensioned corrugated beam is commonly used for road bends where radius of curvature is small as the use of untensioned beam does not require tensioning device. It is designed to deform in beams and to re-direct impacting vehicles on a course as close as possible parallel to the barrier.

**6. When are the conditions which warrant the installation of safety barriers along roads? (BT1)**

In general there are several main conditions which warrant the installation of safety barriers along roads:

(i) It protects vehicles from hitting a roadside object (e.g. bridge pier, sign post, walls etc). Conversely, it protects the roadside object from damage by collision of vehicles.
(ii) It avoids the crossing over of vehicles over central median.
(iii) It protects the vehicle from falling down a steep slope (more than 3m high)
(iv) A poor record of accidents involving run-off vehicles.

**7. Can vehicular parapets withstand the collision of double-decked bus? (VP1)**

Basically the major problem associated with the collision of double-decked bus lies on the possible overturning of the bus upon collision. The overturning moment is the product of impact force and the difference in the centre of gravity of bus and the height of vehicular parapet. The restoring moment is the product of bus weight and 0.5 times the width of bus.

In fact, owing to the elastic deformation of both the parapet and bus, it is expected that the impact force, and hence the overturning moment may not be larger than the restoring moment for 1.1m high vehicular parapet. Computer simulations have to be conducted to verify if a double-decked bus traveling at a certain speed would roll over the parapet when impacted at a certain angle.

**8. Should vehicular parapets be designed to be strong? (VP1)**

Parapets are designed to satisfy different containment levels. The containment level represents the magnitude of impact that the parapet is supposed to uphold.

A parapet designed as low containment level can hardly withstand the impact by large vehicles which may even damage the parapet. On the other hand a parapet designed as high containment level can effectively contain safety large vehicle. However, when it is collided by light vehicles, it is expected that it would cause considerable damage to the light vehicles and its passengers on board. Therefore, strong parapets may not necessarily mean a good parapet.

**9. Is local vehicle parapet strong enough to contain vehicles? (VP1)**

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.

**10. How do steel beam barriers (e.g. tension/untensioned beam barrier and open box barrier) function to contain vehicles upon crashing? (VP1)**

Steel beam barrier consists mainly of horizontal rails and vertical posts. When a vehicle hits the steel beam barrier, the kinetic energy is resolved in three components, namely vertical, normal to barrier and parallel to barrier. The vertical and normal components of kinetic energy are dissipated through deformation and bending of beam and supporting posts. As such, the remaining component (i.e. parallel) guides the vehicle back to the carriageway in a direction parallel to the barrier.

**11. How do noise adsorptive materials function?**

The basic mechanism of noise absorptive material is to change the acoustic energy into heat energy. The amount of heat generated is normally very small due to the limited energy in sound waves (e.g. less than 0.01watts). The two common ways for energy transformation are:

(i) Viscous flow loss
The absorptive material contains interconnected voids and pores into which the sound energy will propagate. As sound waves pass through the material, the wave energy causes relative motion between the air particles and the absorbing material and consequently energy losses are incurred.

(ii) Internal fiction
The absorptive materials have some elastic fibrous or porous structures which would be extended and compressed during sound wave propagation. Other than energy loss due to viscous flow loss, dissipation of energy also results from the internal friction during its flex and squeezing movement.

**12. How do oil interceptors operate? (ORF1)**

Grease and oils are commonly found in stormwater runoff from catchments. They come from the leakage and spillage of lubricants, fuels, vehicle coolants etc. Since oils and grease are hydrocarbons which are lighter than water, they form films and emulsions on water and generate odorous smell. In particular, these hydrocarbons tend to stick to the particulates in water and settle with them. Hence, they should be trapped prior to discharging into stormwater system. Oil interceptors are installed to trap these oil loads coming from stormwater. In commercial areas, car parks and areas where construction works are likely. It is recommended to establish oil-trapping systems in these locations.

Typical oil interceptors usually contain three compartments:

(i) The first inlet compartment serves mainly for the settlement of grits and for the trapping of floatable debris and rubbish.
(ii) The second middle compartment is used for separating oils from runoff.

**13. When should horizontal bars and vertical bars be provided in kerb overflow weirs? (ORF2)**

Overflow weirs should be provided for steep roads (longitudinal gradient>5%) , flat roads (longitudinal gradient<0.5%), sag points and blockage blackspots. For steep roads, flow is rapid and overflow weirs should be provided to accommodate the excess flow. For flat roads, the probability of accumulation of rubbish increases. Therefore, overflow weirs should be provided in these locations to bypass the stormwater flow in case of blockage of gullies caused by trapping of rubbish.

Basically, kerb overflow weirs suffer from the drawback that it provides another passage for debris to enter the gullies and therefore bars (either horizontal or vertical) should be provided to prevent the entry of debris into the weirs. For steep roads, as the main concern is to provide an alternative route for excess flow, horizontal bars should be provided in this case to maintain better drainage efficiency. For flat roads, the purpose of overflow weirs is to trap rubbish and therefore, vertical bars should be provided because it is more effective in prevention of entry of debris [33].

**14. What is the difference between sag gully and on-grade gully? (ORF3)**

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.

**15. Are kerbs necessary in road pavements? (ORF4)**

In general, kerbs are essential in road pavements due to the following reasons (based on Arthur Wignall, Peter S. Kendrick and Roy Ancil):

(i) They provide strength to the sides of road pavements and avoid lateral displacement of carriageway due to traffic loads.
(ii) In terms of road safety, they serve as a separation line between footway and carriageway and aid car drivers in driving safely.
(iii) They act as a vertical barrier to guide the surface runoff collected in road pavements to the gullies.

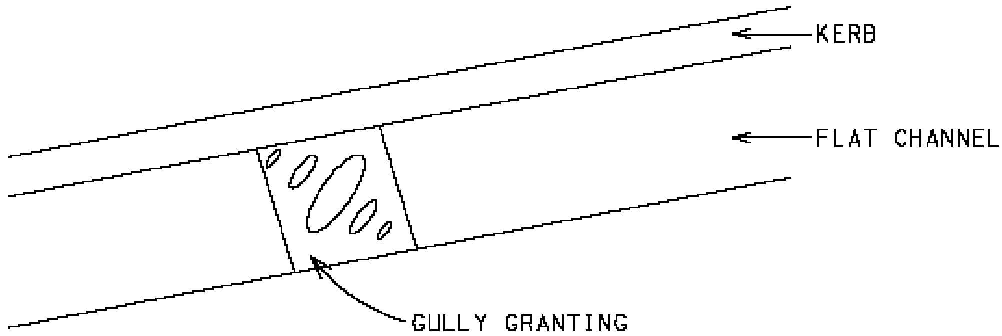



**Fig. Gully grating**

### 16. What is the purpose of road studs in roads? (ORF5)

In nighttime, car drivers could not see where the road ends and where the alignment of road changes in direction. Without sufficient number of road lightings, it is necessary to provide some means to guide the drivers along dark roads. Hence, in 1933 Percy Shaw invented the cat's eye (i.e. road studs) which is based on the principle of cat's eyes. When a ray of light enters the eyes of a cat, the light shall be reflected back towards to the emitting source. As such, with the reflection of car's headlights by road studs, it is possible to identify road conditions and alignment in darkness.

However, the provision of cats' eye is not without problem. For instance, there have been reported accidents arising from loosely installed cat's eye. Some new design of cat's eye uses LEDs which flash at about 100 times per second. However, it is claimed to cause epileptic fits of drivers.

### 17. What is the importance of glass beads in road markings? (ORF6)

Retroreflectivity refers to the part of incident light from headlights of a vehicle being reflected back to the driver. Retroreflectivity is normally achieved in road marking materials by using glass beads or ceramic beads.

These beads are sprayed on the marking materials when the road is marked. The beads are transparent and serve as lenses. When the light passes through the beads, it is refracted through the beads and then reflected back towards the original path of entry. The use of glass beads enhances retroreflectivity which raises the safety level for night driving.

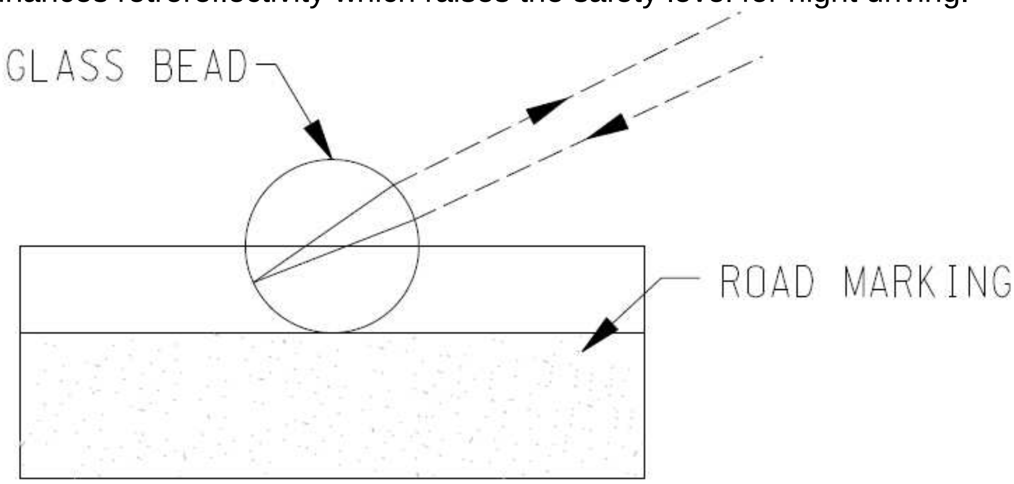



**Fig. Glass bead in road marking**

### 18. Why is hot applied thermoplastic road marking normally used instead of cold applied road paint? (ORF6)

Hot applied thermoplastic road marking appears to be more commonly used than cold applied road paint as road marking materials. For hot applied thermoplastic road marking, it allows the addition of solid glass head which enhances reflectorisation effect. This essentially makes great improvement on the visibility of road markings at night time. Moreover, from past experience the durability of thermoplastic road marking is higher than that of road paint.

### 19. What are the advantages of using rockfill over earthfill to build road embankment?

With the use of rockfill to build embankment, it is not a concern regarding the build-up of pore water pressure during construction so that the embankment can be filled at the faster rate. Moreover, the use of rockfill allows a steeper angle in forming road embankment when compared with earthfill so that it results in a small amount of fill. As such, it helps save the cost of construction.

# 8. Module Seven: Slopes, Excavation and Earthwork

## Objectives

| Element | Description | Objective No. |
|---|---|---|
| **Slope** | | |
| Slope | Janbu's method, Bishop's method and Morgenstern-Price method | S1 |
| | Compaction of fill slope | S2 |
| | Liquefaction of loose fill slope | S3 |
| | Cutting slope | S4 |
| | Landslides by rainfall | S5 |
| | Rockfall | S6 |
| | Reinforced fill | S7 |
| | Decomposed dolerite dykes/ colluvium | S8 |
| Slopes Features | Drainage system and protective covers | SF1 |
| | Erosion control mat | SF2 |
| | Vegetation | SF3 |
| | Sprigging/turfing/hydroseeding | SF4 |
| | Concrete buttress | SF5 |
| | Rock reinforcement | SF6 |
| | Rock dowels and rock bolts | SF7 |
| | Rockfall barrier | SF8 |
| | Horizontal drains | SF9 |
| **Excavation** | | |
| Design | Free earth support method and fixed earth support method | D1 |
| | Rankine's theory of lateral earth pressure | D2 |
| | Peck's pressure envelope | D3 |
| | Loads in struts | D4 |
| Features of Excavation | Diagonal strutting and long flying shores | FE1 |
| | Grout curtain | FE2 |
| | Tiebacks in anchored excavation | FE3 |
| Diaphragm Wall | Function | DW1 |
| | Excess slurry head | DW2 |
| | Bentonite slurry | DW3 |
| | | |

| Element | Description | Objective No. |
|---|---|---|
| **Earthworks and Tests** | | |
| Compaction | Soil compaction test | C1 |
| | Optimum moisture content | C2 |
| | Sand replacement test | C3 |
| | Draglines, backhoes and shovels | C4 |
| **Retaining Walls** | | |
| Retaining Walls | Typical proportioning | RW1 |
| | Shear keys | RW2 |
| | Kicker | RW3 |
| | Granular fill and rock fill | RW4 |
| | Rigid and flexible footings | RW5 |
| | Water control | RW6 |
| | Counterforts in counterfort retaining wall | RW7 |
| | Wires in some gabion walls | RW8 |
| | Arching effect | RW9 |
| **Soil Nails** | | |
| Soil Nails | Mechanism | SN1 |
| | Soil nail head | SN2 |
| | Pull-out test | SN3 |
| | Grout | SN4 |
| **Miscellaneous** | | |
| Miscellaneous | Liquefaction | M1 |
| | Observational Method | M2 |
| | Bentonite slurry and polymeric slurry | M3 |
| | Shotcreting | M4 |
| | Frost heave | M5 |

# Level One (Core FAQs)

## *Part I: Slope*

### 1. Is force and moment equilibrium satisfied by Janbu's method, Bishop's method and Morgenstern-Price method? (S1)

Janbu's method and Morgenstern-Price method are non-circular analytical method and they are frequently used for soil slopes while Bishop's method is circular analytical method. Bishop's Simplified method and Janbu's Simplified method assume that the inter-slice forces are horizontal and inter-slice shear forces are neglected.

| Equilibrium Method | Moment Equilibrium | Force Equilibrium | |
|---|---|---|---|
| | | Horizontal | Vertical |
| Janbu's Simplified | No | Yes | Yes |
| Bishop's Simplified | Yes | No | Yes |
| Morgenstern-Price | Yes | Yes | Yes |

### 2. Why are fill slopes compacted to dense state instead of loose state? (S2)

In rainstorm, the runoff from rainfall infiltrate into the top layer of fill slopes. It may result in saturation of this layer of fills leading to the decrease in soil suction. Consequently shallow slope failure may occur.

If the fill slope is in a loose state, the soils would tend to decrease in volume during deformation. As a result this induces a rise in pore-water pressure which triggers slope failure in form of mud-avalanche.

If the fill slope is in a dense state, the soils would tend in increase in volume during deformation and it only fails like a mud slump.

### 3. Other than liquefaction, what are the possible causes of failure of loose fill slopes? (S3)

Other than static liquefaction, slow-moving slips driven by transient pore water pressure leading to high speed landslide are the other possible cause of failure of loose fill slopes.

For loose fill lying on low permeability soil layers, there is potential storage of infiltrating water when the slope of underlying low-permeability soil layer is mild. As such, there is a localized zone of high transient pore water pressure induced within the fill material. Flowslides normally start with a local slip caused by transient pore water pressure by soil layering or flow restriction. Then, the nature of slow-moving soil debris and the geometry of slip result in a fast landslide.

**4. 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.

**5. 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.

**6. Does cutting slope cause slope deformation or slope failure? (S4)**

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.

**7. How are landslides triggered by rainfall? (S5)**

After rainfall, groundwater pressure is built up and this elevates the ground water table. The water inside the pores of soil reduces the effective stress of soils. Since shear strength of soils is represented by the following relations:

Shear strength = cohesion + effective stress x tanΦ where Φ is the friction angle of soils

Hence, the presence of water causes a reduction of shear strength of soils and this may lead to landslide. On the other hand, the rainfall creates immediate instability by causing erosion of slop surface and results in shallow slope failure by infiltration. In addition, the rain may penetrate slope surface openings and forms flow paths. As a result, this may weaken the ground.

**8. Why do landslides occur though the rainfall has not led to full saturation in the sliding zone? (S5)**

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.

**9. What are the major causes of rockfall? (S6)**

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.

**10. What are the functions of drainage system and protective covers of slopes? (SF1)**

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.

**11. What is the purpose of installation of erosion control mat in slopes? (SF2)**

Steep slopes are prone to intermittent high velocity flows during rainstorm and this causes erosion at slope surface which prevents the growth of vegetation.

Erosion control mat is installed to control soil erosion and provide soil stability until vegetation can be established. The principal function of erosion control mat is to prevent pre-vegetated soil loss by stabilizing and protecting soils from rainfall and surface erosion. Moreover, it could provide a long-term artificial erosion control system which would increase the shear resistance of vegetation and provide long-term, tenacious reinforcement of the root system.

**12. What is the reason of adding steel wire mesh when using erosion control mat in slopes? (SF2)**

It is not uncommon that the use of erosion control mat in slopes is accompanied by the addition of steel wire mesh. The system of erosion control mesh with a steel wire mesh proves to be a more effective method to control surface erosion. In case surface erosion occurs even in the presence of erosion control mat, the soil debris could be trapped between the steel wire mesh and the slope surface. As such, the steel wire mesh essentially serves as an additional protective layer to avoid further occurrence of erosion.

**13. How can vegetation improve soil shear strength and slope stability? (SF3)**

Vegetation in slopes could modify the soil moisture condition in two ways:

(i) Soil moisture is absorbed by roots of vegetation and the water is transpired to its root;
(ii) It intercepts the rainfall by reducing the amount of rainwater penetrating into the slope.

Both processes, i.e. evapotranspiration and interception effected by the presence of vegetation contribute to dry soil conditions. Moreover, evapotranspiration results in soil suction in unsaturated soil slopes, which further improves the shear strength of soil. As such, it tends to delay the time of saturation in slopes and this essentially improves the slope stability.

**14. What is the difference between sprigging and turfing? (SF4)**

A sprig is a part of stem with crowns and roots which is cut from a rhizome or stolon. Sprigging is a type of vegetative planting by placing sprigs in spaced intervals in holes. With proper transplantation and maintenance, a sprig is capable of growing as a plant. The cost of sprigging is moderate and vegetation is expected to grow for around six months. However, this method requires high manual input and is labour intensive.

Turfing is the direct application of vegetation with developed roots. The cost of turfing is high when compared with sprigging. However, it is the quickest way to get vegetation. Within a month mature-looking vegetation is ready for use.

**15. What is the difference between hydroseeding and turfing on slopes? (SF4)**

Hydroseeding is the provision of grass seed mixed with fertilizer and nutrient by spraying. The grass seed shall grow and the root of grass serves as reinforcing fiber to hold tightly the surface soils.

Turfing is the direct application of grass with developed roots on slope surface. The already-developed grass is expected to grow easier when compared with hydroseeding. The roots of grass shall extend onto soils to strength the overall surface.

**16. How can concrete buttress stabilize rock fall?(SF5)**

Sometimes there is a cavity in rock slopes which is caused by previous rock falls. Concrete buttress, together with the use of rock dowels, could help to stabilize the rock slope.

The buttress serves to avoid local toppling of rock face. Moreover, it keeps protects the unstable rock and keep it in place. The presence of buttress also helps hold up the overhang. The size of buttress should be large enough to support the weak rock. In case the surface on which the buttress is founded is not normal to the resultant forces exerting on the buttress, buttress is tied to rock surface by using rock dowels to avoid the occurrence of sliding.

**17. How does rock reinforcement function? (SF6)**

For rocks with widespread fractures, individual blocks resulting from these fractures may fall out and as a result slope failure may occur. Rock reinforcement (e.g. rock dowels, bolts or anchors) is installed to bolt through the discontinuities in rock to enhance the rock to behave as a single unit. With the bolting across block interfaces, the stresses would be altered within the rock mass. For untensioned rock dowels, they may be subjected to tensile forces arising from rock movement. Other than the provision of rock reinforcement, shotcreting is another method to reinforce the rock. It functions by griping the rock together and maintaining the small blocks which hold the large blocks in position [37].

**18. Do inclined rock bolts perform better than rock bolts normal to joints? (SF7)**

The use of rock bolts can be dated back to Roman Empire and it was commonly believed that rock bolts pin surface rock (bedded rock strata or individual rocks) to more stable rock. With the insertion of rock bolt into rock slope, it increases the rock mass's stiffness and strength with respect to shear and tensile loads. Axial force in rock bolts consist of a force component normal to joint plane which contributes frictional resistance, and a force component parallel to joint plane which contributes to dowel action.

Inclined rock bolts helps to stiffen shear and results in an increase in shear strength at smaller displacements. Rock bolts perpendicular to shear planes provide the lowest shear resistance. The optimal inclination of rock bolt is $30^o$ to $60^o$ based on past experimental works.

**19. What are the differences between rock dowels and rock bolts? (SF7)**

Rock dowels are passive reinforcing elements which need some ground displacement for activation. In installation of rock dowels, a hole is drilled and untensioned steel bars are inserted into the hole. When displacements along joints occur, rock dowels are subject to both shear and tensile stresses. The level and ratio of shear and tensile stress depends on the properties of the surrounding ground, the grout material filling the annular gap between the dowel and the ground and the strength and ductility parameters of the rock dowel itself. Moreover, the degree of dilation during shear displacement affects the level of stress acting within the dowel.

Rock bolts are tensioned once the anchorage is attained to actively set up a compressive force into the surrounding rock. This axial force increases the shear capacity and is generated by pre-tensioning of the bolt. The system requires a bond length to enable the bolt to be tensioned. In essence, rock bolts start to support the rock as soon as they are tensioned and the rock does not have time to start to move before the rock bolt becomes effective.

**20. Why does flexible wire rope nets effective in stopping rockfall? (SF8)**

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 boulders.
(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.

**21. Will the posts of rockfall barrier be damaged when it is hit directly by rock boulders? (SF8)**

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.

**22. What are the potential problems of using clayey backfill in reinforced fill? (S7)**

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.

**23. How does geogrid function in reinforced fill? (S7)**

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.

**24. 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.

**25. How can decomposed dolerite dykes affect slope stability? (S8)**

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.

**26. Does the presence of colluvium beneficial to slopes? (S8)**

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.

**27. How do horizontal drains help to stabilize slopes? (SF9)**

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.

**28. Is bleeding test an essential requirement for grout?**

Bleeding is a form of segregation in which a layer of water migrates to the surface of the grout during the initial stage of cement hydration process. Later on, some of the floating water is re-absorbed into the grout due to further hydration reactions. Even without the problem of bleeding, there is a total reduction of volume of grout after hydration action when compared with the total initial individual volume of cement and reacted water. Bleeding tests should be carried out for grout because of the following reasons [22]:

(i) During bleeding, the upflow of water from grout mixture leads to the formation of channel paths inside the grout mix. These channels act as potential paths for aggressive materials to pass through as these channels would not be closed during further hydration of the grout.
(ii) The loss in volume by bleeding generates voids inside the grout mix which affects the properties and performance of the grout. Moreover, it increases the chance of corrosion of steel elements protected by the grout. (e.g. tendons)
(iii) In bleeding test, there is a usual requirement of total re-absorption of water after 24 hours of grout mixing because for some cold countries, this layer of water may cause severe freezing problem leading to frost damage.

# Level One (Core FAQs)

## ***Part II: Excavation***

### 1. What is the difference between free earth support method and fixed earth support method? (D1)

For free earth support method, the soils at the lower part of piling is incapable of inducing effective restraint so that it would not result in negative bending moments. In essence, the passive pressures in front of the sheet piles are insufficient to prevent lateral deflection and rotations at the lower end of piling. No passive resistance is developed on the backside of the piling below the line of excavation.

For fixed earth support method, the piling is driven deep enough so that the soil under the line of excavation provides the required restraint against deformations and rotations. In short, the lower end of piling is essentially fixed.

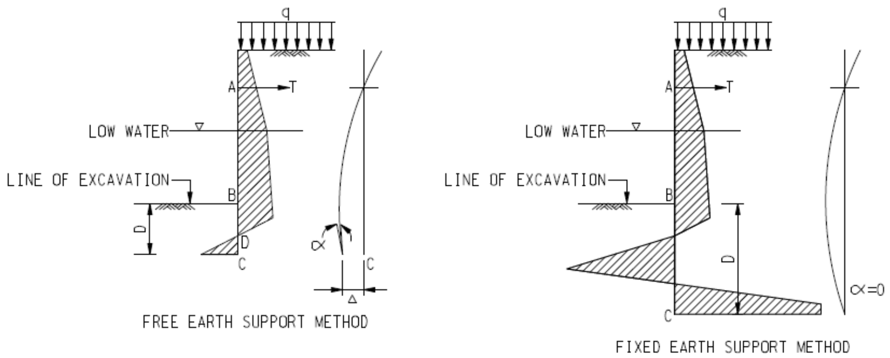



**Fig. Free earth support method and fixed earth support method**

### 2. In braced excavation, why is Rankine's theory of lateral earth pressure not applicable? (D2)

In braced excavation, sheetpiling is constructed at the first place, followed by the installation of struts as excavation proceeds. Following the installation of first row of struts, the depth of excavation is small so that there is no major yielding of soils. However, as further excavation takes place, soils yield before the installation of $n^{th}$ row of struts. The first row of struts prevents yielding near the ground surface. As such, deformation of wall increases with depth with the smallest at the ground level. Owing to the effect of construction method of braced excavation, it differs from the deformation condition of Rankine's theory. This is attributed to arching effects in which there exists upward redistribution of loads. The upper part of braced excavation is in the state of elastic equilibrium while the lower part is in the state of plastic equilibrium.

**3. For Peck's pressure envelope for braced excavation, should total weight or effective weight be used in rectangular and trapezoidal envelope? (D3)**

The use of active and at-rest theory is not applicable in braced excavation. In essence, upper struts tend to be more heavily loaded while lower struts appear to be less loaded when compared with active pressure theory.

Peck then measured the bracing loads which were converted back to soil pressures. For example, the pressure envelope for non-cohesive soils is $0.65rHK_a$ ($r$=soil density, H=height of excavation and $K_a$=active pressure coefficient).

Some engineers consider *r* as total soil weight without applying any water pressure. However, Peck has said "the earth pressures are essentially effective active pressures multiplied by a factor and redistributed as a rectangle or a trapezoid." Hence, effective weight of soils should be used for *r* with water pressures added separately.

**4. Should high preloads be adopted in struts in braced excavation? (D4)**

Preloading in struts in braced excavation helps eliminate the potential movement. The application of preload decreases the shear stress in soils established previously by excavation. As a result of stiffening of soils, the soil movement is declined accordingly.

However, the use of very high preloads in struts may not be desirable because the local outward movement at struts may cause damage to nearby utilities.

**5. What are the applications of diagonal strutting and long flying shores in braced excavation? (FE1)**

Diagonal strutting is sometimes used at the corners of excavation to leave a large working space at excavation level.

Sometimes, raking struts are observed in long flying shores across excavation and these struts serve to decrease the span length of struts.

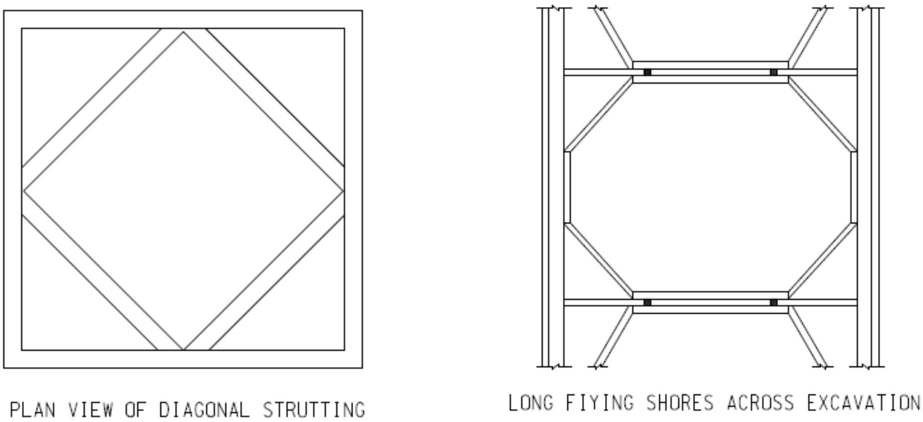



**Fig. Diagonal strutting and long flying shores in braced excavation**

**6. What is the purpose of installation of "grout curtain" around excavation? (FE2)**

When excavation work is carried out in grounds with highly permeable soils, other than the installation of well points to lower down the groundwater table, consideration may be given to the injection of grout to the soils [60]. The purpose of the injection of grout is to fill the pore spaces and cavities of soils with grout and to reduce the permeability of soils. The method of grouting is effective in coarse soils but not for sands. In essence, "grout curtain" is constructed around the excavation by installation of several rows of injection holes for grouting.

**7. What is the significance of free length and fixed length in tiebacks in anchored excavation? (FE3)**

The use of tiebacks in deep excavation allows uninterrupted earth moving within the excavation zone owing to the absence of interior obstructions. The spacing of tieback should not be placed too close as this may impair the capacity of tieback because of the interference between adjacent grouted zones.

A tieback is made first by drilling a hole by a drill rig, followed by placing a bar in the drilled hole. Concrete is then poured in the hole and the connection of tieback with wall is made lastly. A tieback anchor consists of an anchorage located in a bearing layer and the anchor is tensioned at the front face of the wall. The portion of the anchor which transmits the force to the surrounding soil is called the "fixed length". One the other hand, the "free length" of tieback transfers the force from the fixed length through the anchor head to the wall.

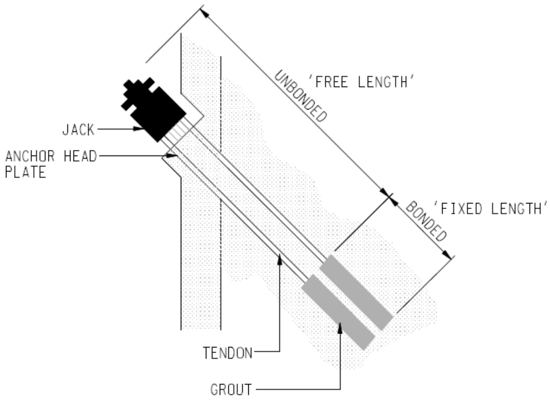



**Fig. Free length and fixed length in tieback**

### 8. What are the functions of diaphragm walls? (DW1)

The functions of diaphragm walls are as follows:

(i) It is designed to retain soils during the construction of underground structures.
(ii) It helps to control the movement of ground during construction.
(iii) It is intended to take up high vertical loads from aboveground structures during construction (e.g. top-down approach). In addition, during the servicing of the completed structures, the diaphragm walls, internal piles and basement raft act together as a single unit to perform as piled raft.

### 9. Why should excess slurry head be maintained in diaphragm wall? (DW2)

For the construction of diaphragm walls adjacent to buildings, previous experience showed that excess slurry head above groundwater level had to be maintained to limit the ground settlements during the construction of diaphragm walls. In fact, the excess slurry head can be achieved by the following methods. The first one is to construct a ring of well points to lower the piezometric level to achieve a higher excess slurry head in diaphragm walls. Alternatively, guide walls may be raised above ground level to accommodate the slurry column.

**10. Why is bentonite slurry commonly used in diaphragm wall construction? (DW3)**

Bentonite slurry is one of the most common excavation fluid used in constructing diaphragm wall. Bentonite clay (in powder form) and water are combined in a colloidal mixer and clay particles bond to each other and set to form a gel when left to stand for a period of time. When the bentonite is set in motion, it reverts back to the fluid state rapidly.

Bentonite slurry shores the trench to stabilize the excavation and forms a filter cake on the slurry trench walls that reduces the slurry wall's final soil permeability and to reduce ground water flow. The gel strength and viscosity properties of the bentonite clay allow for cutting suspension and removal.

**11. Should high density or low density bentonite slurry be used in diaphragm wall construction? (DW3)**

The use of high density bentonite slurry could improve trench stability. It helps to retain cuttings and particles in suspension and reduce the loss of bentonite slurry into soils of high permeability such as sand. However, the use of low density bentonite slurry is desirable from operation point of view. Low density bentonite slurry tends to be more neatly displaced from soils and reinforcement. Moreover, the pumping of low density bentonite slurry is more easily to be carried out.

**12. What is the difference in mechanism in resisting clay and normal soils by bentonite slurry in diaphragm wall construction? (DW3)**

For normal soils, water in bentonite slurry penetrates into the sandy walls and leaves behind a layer of bentonite particles on the surface of the soils. The bentonite particles form the filter cake of low permeability on the excavated faces. The filter would be formed only when slurry pressure is greater than the pore water pressure in excavated soils. The filter cake serves as impermeable layer and allows the application of full hydrostatic pressure of bentonite slurry on the excavated surface of soils.

For soils with low permeability such as clay, there is little water passage from slurry to excavated clay surface so that filter cake would hardly be formed. As a result, slurry pressure simply applies on clay surface.

# Level One (Core FAQs)

## ***Part III: Earthwork and Tests***

### 1. Do soil compaction test results over 100% mean over-compaction? (C1)

Soils can experience over-compaction if the compactor makes too many passes over it. In fact, relative soil compaction test results over 100% do not necessarily mean over-compaction because the relative compaction is based on the maximum dry density of the soil obtained by the Proctor test and this does not necessarily refer to *absolute* maximum dry density.

Over-compaction is considered undesirable because it may eventually create cracks in the underlying compacted material so that it results in a decrease in density. Moreover, it causes waste of machine power and manpower which is undesirable.

### 2. In soil compaction test, if a test result exceeds 100%, should engineers accept the result? (C1)

Soil compaction is the process of increasing the soil density by reducing the volume of air within the soil mass.

Soil compaction depends mainly on the degree of compaction and the amount of water present for lubrication. Normally 2.5kg rammers and 4.5kg rammers are available for compaction in laboratories and the maximum dry densities produced by these rammers cover the range of dry density obtained by in-situ compaction plant.

Regarding the second factor of water content, it affects the compaction in the following ways. In low water content, the soils are difficult to be compacted. When water content is increased gradually, water will lubricate the soils and this facilitates the compaction operation. However, at high water content, as an increasing proportion of soils is occupied by water, the dry density decreases with an increase in water content.

For soil compaction tests, the dry density obtained from compaction carried out in-situ by vibrating roller/vibrating plate is compared with the maximum dry density conducted in laboratories using 2.5kg rammer of compaction with similar soils. In essence, the in-situ compaction is compared with the compacting effort of using 2.5kg (or 4.5kg) rammer in laboratories. In case the compaction test results indicate values exceeding 100%, it only means that the in-situ compaction is more than that being carried out in laboratories which is treated as the basic criterion for satisfactory degree of soil compaction. Therefore, the soil results are acceptable in case compaction test results are over 100%. However, excessive compaction poses a risk of fracturing granular soils resulting in the reduction of soil strength parameters.

**3. Are there any differences in the methods of compaction between clayey soil material and sandy material?**

As suggested by Lars Forssblad (1981), the three main actions of compaction are static pressure, impact force and vibration. Different compactors contain one or more modes of these actions. For example, vibratory tampers perform mainly by the principle of impact while vibratory rollers work with principle of static pressure and vibration.

For sandy soils, vibration is adequate for normal compaction because the action of vibration sets the soil particles in motion and friction forces between soil particles are virtually demolished. During this vibration motion, the soil particles rearrange themselves to develop a dense state.

For normal soils, it is necessary to combine the action of vibration together with static pressure to breakdown the cohesion forces between soil particles in order to allow for better compaction. The static pressure of vibratory machines is adopted to exert a shearing force to eliminate the cohesion in clayey soils.

**4. For compaction of free-draining sands or gravels, what is the optimum moisture content to achieve maximum density? (C2)**

The compaction curve of sandy materials is totally different from that of clayey materials. For sands or gravels, there are two situations of maximum density, namely the completely dry condition and the complete water saturation. For moisture content of sands and gravels between these two states, the dry density obtained is lower than that obtained in the above-mentioned states. The presence of capillary forces account for the difficulty of compaction sand at water contents between virtually dry and saturated state. They are formed in partially filled water void between soil particles and perform as elastic ties cementing soil particles together. Reference is made to Lars Forssblad (1981).

The compaction curve for clay is suitable for the majority of soil types except sands and gravels because a small amount of clay in soils is sufficient to make the soils impermeable.

**5. Is sand cone (replacement) test suitable for all soils? (C3)**

Sand cone (replacement) test is normally carried out to determine the in-situ and compacted density of soils. This testing method is not suitable for granular soils with high void ratio because they contain large voids and openings which provide an access for sand to enter these holes during the test. Moreover, soils under testing should have sufficient cohesion so as to maintain the stability of the sides of excavation during the excavation step in sand cone (replacement) test. In addition, organic or highly plastic soils are also considered not suitable for this test because they tend to deform readily during the excavation of holes and they may be too soft to resist the stress arising from excavation and from placing the apparatus on the soils.

**6. What are the different applications of draglines, backhoes and shovels? (C4)**

An excavator is defined as a power-operated digging machine and it includes different types like shovels, draglines, clamshells, backhoes, etc.

A dragline possesses a long jib for digging and dumping and it is used for digging from grade line to great depths below ground. Its characteristic is that it does not possess positive digging action and lateral control of normal excavators. A dragline is normally deployed for bulk excavation.

A backhoe is designed primarily for excavation below ground and it is especially employed for trench excavation works. It digs by forcing the bucket into soils and pulling it towards the machine and it possesses the positive digging action and accurate lateral control.

A shovel is a machine that acts like a man’s digging action with a hand shovel and hence it is called a shovel. It digs by putting the bucket at the toe of excavation and pulling it up. Though a shovel has limited ability to dig below ground level, it is very efficient in digging above ground like digging an embankment.

# Level One (Core FAQs)

## ***Part IV: Retaining Walls***

### 1. What is the typical proportioning of a retaining wall? (RW1)

The base slab thickness of a cantilever retaining wall is about 10% of the total height of retaining wall. The length of base slab is about 50-70% of the total height of retaining wall. Generally speaking, the thickness of wall stems may vary along the stem provided that its size should not be less than 300mm to facilitate concrete placement.

For retaining wall with a total height exceeding 8-12m, it is recommended to adopt counterfort retaining wall. The counterforts in counterfort retaining wall are normally spaced at about 30% to 70% of the total height. The length of base slab is about 40-70% of the total height of retaining wall.

### 2. What is the function of shear keys in the design of retaining walls? (RW2)

In determining the external stability of retaining walls, failure modes like bearing failure, sliding and overturning are normally considered in design. In considering the criterion of sliding, the sliding resistance of retaining walls is derived from the base friction between the wall base and the foundation soils. To increase the sliding resistance of retaining walls, other than providing a large self-weight or a large retained soil mass, shear keys are to be installed at the wall base. The principle of shear keys is as follows:

The main purpose of installation of shear keys is to increase the extra passive resistance developed by the height of shear keys. However, active pressure developed by shear keys also increases simultaneously. The success of shear keys lies in the fact that the increase of passive pressure exceeds the increase in active pressure, resulting in a net improvement of sliding resistance.

On the other hand, friction between the wall base and the foundation soils is normally about a fraction of the angle of internal resistance (i.e. about $0.8\varphi$ ) where $\varphi$ is the angle of internal friction of foundation soil. When a shear key is installed at the base of the retaining wall, the failure surface is changed from the wall base/soil horizontal plane to a plane within foundation soil. Therefore, the friction angle mobilized in this case is $\varphi$

instead of 0.8φ in the previous case and the sliding resistance can be enhanced.

**3. Where is the best position of shear keys under retaining walls? (RW2)**

The installation of shears keys helps to increase the sliding resistance of retaining walls without the necessity to widen the their base. The effect of shears keys enhances the deepening of the soil failure plane locally at the keys. The increased sliding resistance comes from the difference between the passive and active forces at the sides of the keys. In case weak soils are encountered at the base level of shear keys, the failure planes along the base of retaining walls due to sliding may be shifted downwards to the base level of the keys.

Shear keys are normally designed not to be placed at the front of the retaining wall footing base because of the possible removal of soils by excavation and consequently the lateral resistance of soils can hardly be mobilized for proper functioning of the shear keys [30]. For shear keys located at the back of footings, it poses a potential advantage that higher passive pressures can be mobilized owing to the higher vertical pressure on top of the passive soils.

**4. Why is the kicker of reinforced concrete cantilever retaining walls located at the position of largest moment and shear force? (RW3)**

Normally for reinforced concrete cantilever retaining walls, there is a 75mm kicker at the junction wall stem and base slab to facilitate the fixing of formwork for concreting of wall stems. If a higher kicker (i.e. more than 75mm height) is provided instead, during the concreting of base slab the hydraulic pressure built up at kicker of fresh concrete cause great problem in forming a uniform and level base slab.

Despite the fact that the position of kicker in a cantilever retaining wall is the place of largest flexure and shear, there is no option left but to provide the kicker at this position.

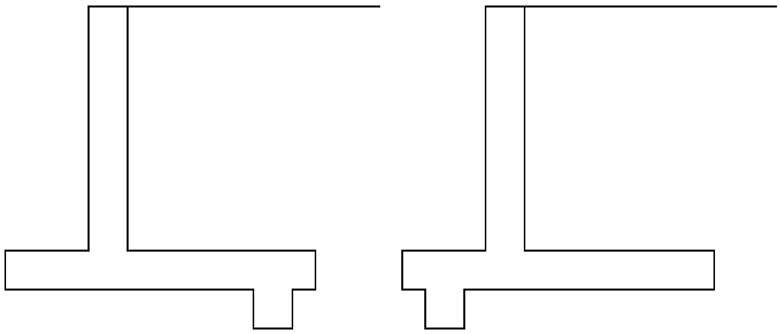



**Fig. Different locations of shear key in retaining wall**

### 5. Are layers of granular fill and rock fill essential at the base of concrete retaining walls? (RW4)

It is not uncommon that granular fill layers and rockfill layers are placed beneath the bottom of concrete retaining walls. The purpose of such provision is to spread the loading in view of insufficient bearing capacity of foundation material to sustain the loads of retaining walls. Upon placing of granular fill layers and rockfill layers, the same imposed loads are supported by a larger area of founding material and hence the stress exerted by loads is reduced accordingly.

Layers of granular fill and rockfill materials are not standard details of concrete retaining wall. If we are fully satisfied that the founding material could support the loads arising from retaining walls, it is not necessary to provide these layers of granular fill and rockfill materials.

### 6. How does pressure distribution vary under rigid and flexible footings? (RW5)

For thick and rigid footings, the pressure distribution under the footings is normally assumed to be linear. If uniform and symmetrical loadings are exerted on the footings, the bearing pressure is uniformly distributed. However, if unsymmetrical loads are encountered, then a trapezoidal shape of bearing reaction would result.

For flexible footings on weak and compressible soils, the bearing pressures under footing would not be linear. As such, a detailed investigation of soil pressures is required in order to determine the bending moment and shear forces of the structure.

**7. What is the function of mortar in brick walls?**

A typical brick wall structure normally contains the following components:

(i) a coping on top of the brick wall to protect it from weather;
(ii) a firm foundation to support the loads on the brick wall; and
(iii) a damp course near the base of the brick wall to avoid the occurrence of rising damp from the ground.

Bricks are bedded on mortar which serves the following purposes [66]:

(i) bond the bricks as a single unit to help resist lateral loads;
(ii) render the brick wall weatherproof and waterproof; and
(iii) provide even beds to enhance uniform distribution of loads.

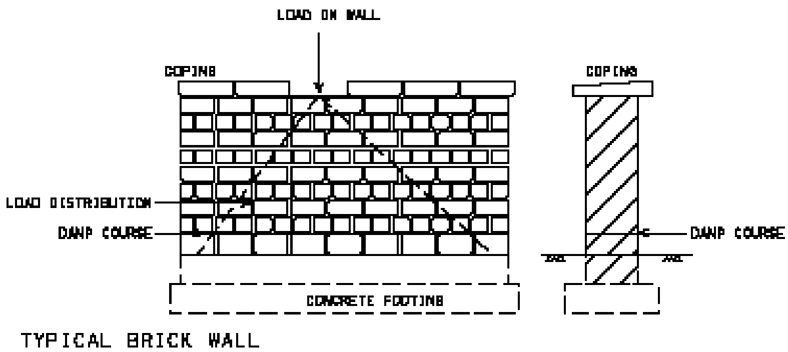



**Fig. Brick wall**

**8. How could counterforts in counterfort retaining wall assist in resisting earth pressure? (RW7)**

Counterforts are used for high walls with height greater than 8 to 12 m. They are also used in situations where there is high lateral pressure, i.e. where the backfill soils are heavily surcharged.

The counterforts tie the base slab and wall stem together and they act as tension bracing which strengthen the connection between wall and base slab. The counterforts help to reduce bending moment and shear forces induced by soil pressure to the retaining wall. Moreover, it also serves to increase the self-weight of the retaining wall which adds stability to the retaining wall.

**9. How to carry out water control for sheet pile walls? (RW6)**

Ground water flow into excavations constructed by sheet pile walls should be minimized in order to save the cost of the provision of pumping systems or well points to lower the water table inside the excavation. In case a layer of impermeable material like clay is located slightly below the excavation, it may be desirable to drive the sheet piles further into this layer and the cost of further driving may be less than the cost of the provision of continuous pumping in the excavation. On the other hand, if there is no impermeable layer beneath the excavation, engineers may consider driving the sheet piles further so as to increase the flow path of groundwater into the excavation and this helps to reduce the amount of water flow into the excavation. Similarly, a cost benefit analysis has to be carried out to compare the extra cost of driving further the sheet piles with the reduced pumping costs.

**10. Why are the wires in some gabion walls designed in hexagonal patterns with doubly twisted joints? (RW8)**

Gabions are wire mesh boxes which are filled with stones and they are placed in an orderly pattern to act as a single gravity retaining wall. Lacing wires or meshes are designed to hold the gabion boxes together. Most of the wires are zinc-coated or PVC coated to prevent the steel wire from corrosion. Moreover, it is common that the wires are fabricated in hexagonal patterns with doubly twisted joints to avoid the whole gabion mesh from disentanglement in case a wire accidentally breaks. Owing to the nature of gabion filling materials, they are very permeable to water. They have particular application in locations where free water drainage has to be provided. Moreover, gabions are capable of accommodating larger total and differential settlements than normal retaining wall types so that they are commonly found in locations where the founding material is poor.

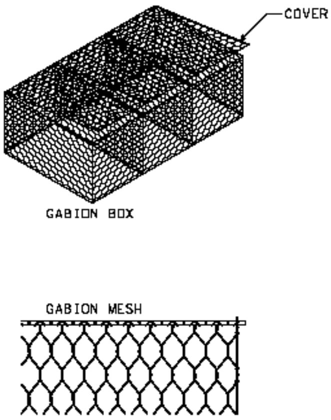



**Fig. Wires of gabion walls**

## 11. What is arching effect in soils? (RW9)

Arching occurs when there is a difference of the stiffness between the installed structure and the surrounding soil. If the structure is stiffer than the soil then load arches onto the structure. Otherwise, if the structure is less stiff than the soil then load arches away from the structure.

For instance, if part of a rigid support of soil mass yields, the adjoining particles move with respect to the remainder of the soil mass. This movement is resisted by shearing stresses which reduce the pressure on the yielding portion of the support while increasing the pressure on the adjacent rigid zones. This phenomenon is called the arching effect.

The principle of soil arching can be easily illustrated by buried pipes. If a rigid pipe is installed in soils, soil columns on both sides of the rigid pipe are more compressive than the soil columns on top of the rigid pipe because of the higher stiffness of rigid pipes when compared with soils. As such, soil columns on both sides tend to settle more than the soils on top of the rigid pipe and this differential settlement causes a downward shear force acting along the sides of soil columns on top of the rigid pipe. As such, the load on the rigid pipes becomes larger than the sole weight of soil columns on its top. Similarly, if a flexible pipe is adopted instead, the above phenomenon shall be reversed.

# Level Two (Advanced FAQs)

## ***Part I: Soil Nails***

### 1. What is the basic mechanism of soil nails in improving soil stability? (SN1)

Soil nails improve slope stability by:

(i) Partial increase in the the normal force and shear resistance along potential slip surface in frictional soils.
(ii) Direct reduction of driving force along potential slip surface for cohesive and frictional soils.
(iii) Soil nail head and facing provides containment effect to limit the deformation near slope surface.

Soil nail head serves to provide reaction in mobilizing tensile force in soil nail. Moreover, it provides confinement to soils in active zone behind soil nail head and avoids the occurrence of local failure between adjacent soil nails.

### 2. Is soil nail head an essential feature of soil nails? (SN2)

Soil nail head is an essential feature of soil nails which help prevent active zone failure. Active zone failure involves shallow failure in the outer part of slope within active zone and it fails by separating and sliding down the slopes and sols nails remain in place.

Soils nail heads also enhances the mobilization of tensile resistance of soil nails and increases the factor of safety of slopes.

### 3. What are the main reasons for conducting pull-out tests for soil nails? (SN3)

There are mainly four reasons for this test:

(i) To check and verify the bond strength between soil and grout adopted during the design of soil nails. This is the main objective of conducting soil nail pull-out test.
(ii) To determine the bond strength between soil and grout for future design purpose. However, if this target is to be achieved, the test nails should be loaded to determine the ultimate soil/grout bond with a

upper limit of 80% of ultimate tensile strength of steel bars.
(iii) To check if there is any slippage or creep occurrence.
(iv) To check the elastic and plastic deformations of test nails. This is observed during the repeated loading and unloading cycles of soil nails.

Note: Pull-out tests are carried out by applying specified forces in an attempt to pull out the constructed soil nails.

**4. What is the purpose of loading and unloading cycles in pull-out tests of soil nails? (SN3)**

In carrying out pull-out tests for soil nails, it normally requires the loading and unloading of soil nails of several cycles up to 80% of ultimate tensile strength of soil nails. The principal function of soil nail tests is to verify the design assumptions on the bond strength between soil and grout which is likely to exceed the design values based on past experience. In addition, the ultimate bond strength between soil and grout can be determined and this information is helpful as a reference for future design.

Then someone may query the purpose of conducting load/unloading cycles of soil nails as it does not provide information on the above two main purposes of soil nails. In fact, loading and unloading soil nails can provide other important information on their elastic and plastic deformation behaviour. However, as stress levels in soil nails are normally low, the knowledge on elastic and plastic performance may not be of significant value. On the other hand, the creep and slippage performance of soils nails can also be obtained which may be useful for some soils.

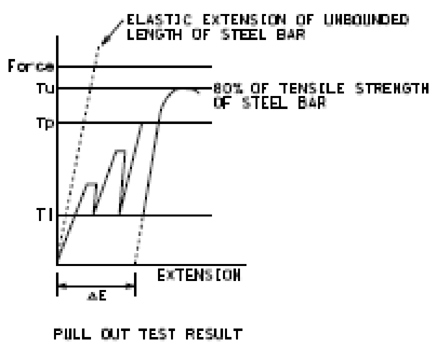



**Fig. Typical pull-out test result**

**5. What is the relation of bearing pressure on soil nail head to the ratio $L_a/L_b$, where $L_a$ is the length of soil nail before the potential slip circle while $L_b$ is the length of soil nail beyond the potential slip circle? (SN2)**

The unstable soil mass before the potential circular slip is resisted by two components: soil nail head bearing pressure and friction of soil nail in the unstable soil mass. Therefore, the longer is the length of soil nail before potential slip circle $L_a$, the higher is the proportion of forces being resisted by frictional forces and hence the smaller amount is to be resisted by soil nail head. Hence, the smaller the ratio $L_a/L_b$, the greater is the resistance provided by soil nail head.

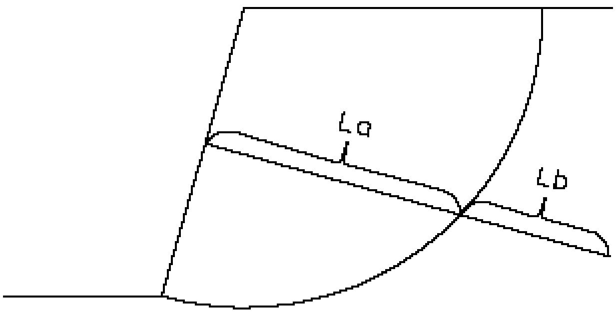



**Fig. $L_a$ & $L_b$ in slopes**

**6. Soil nails are mainly designed for stabilization of major slips. How should designer cater for the stability of minor slips?**

There are some methods to treat minor slips:
(i) Adoption of smaller diameter size bars at closer spacing;
(ii) Installation of tie beams at the same horizontal levels;
(iii) Provision of steel wire meshes in-between soil nails; and
(iv) Provision of short soil nails in combination of long soil nails.

**7. How do soil nails help to improve the stability of slopes? (SN1)**

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.

**8. Why are steeply inclined soil nails not commonly used in stabilizing slopes? (SN1)**

Based on stress-strain relations, when the angle of soil nails below the horizontal (angle b in the figure below) is small, the soil nails intersect the normal to the potential slope failure surface at a relatively large angle. Tensile stresses can be readily developed in the soils nails. However, if the angle of soil nails below the horizontal is large, the soil nails intersect the normal to the potential slope failure surface at a small angle and little or no tensile stress would be mobilized in the soil nails. In this case, it is ineffective in utilizing its tensile strength to prevent slope failure. In the worst scenario, the soils nail would under the state of compression for even steeper soil nails.

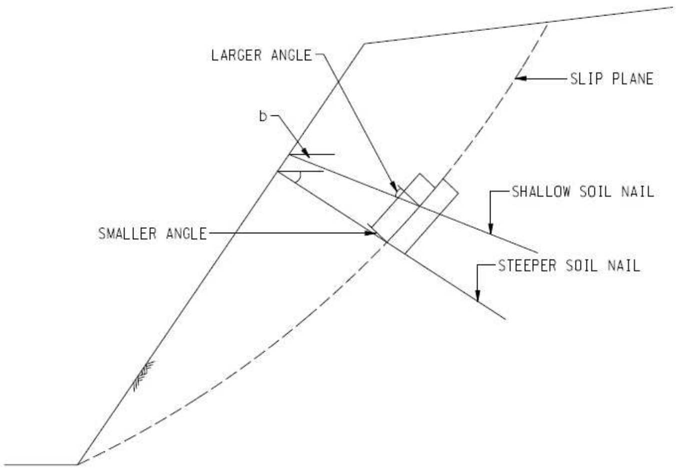



**Fig. Different inclination of soil nails**

**9. Can grout be utilized in providing tensile resistance of soil nails? (SN4)**

The passive nature of soils nails requires a small movement for the nails to take up loading. During this process, it is understood that the grout annulus around the nail would crack to allow for these small displacements.

Therefore, the tensile capacity of grout is normally ignored in design and only compressive capacity might be considered.

**10. Can sheets or geo-grids replace reinforcing elements in soil nails?**

Where soil nails are intended for improving the slope stability of existing ground, sheets or geo-grids can hardly replace reinforcing elements in soil nails. Practically speaking, the reinforcing of existing slopes limits the types of reinforcing elements to be adopted. For instance, sheets or geo-grids do not have sufficient bending stiffness to be inserted into exiting slope and they are usually placed in soils as soil layers are built up. The reinforcing element of exiting ground requires steel bars with good tensile strength.

# Level Two (Advanced FAQs)

## ***Part II: Miscellaneous***

### 1. Does liquefaction occur to sand only? (M1)

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? (M1)

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 are the rationales of Observational Method in geotechnical works? (M2)**

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.

**4. 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.

**5. Which one is better, bentonite slurry and polymeric slurry? (M3)**

For the construction of diaphragm walls, bentonite slurry is commonly used to form a filter cake on walls of trenches to support earth pressure. The use of bentonite solely is based on its thixotrophic gel viscosity to provide support.

Though the cost of polymer is generally more expensive than bentonite, the use of polymer is increasing because polymer is generally infinitely re-usable and very small amount of polymer is normally required for construction works. The disposal cost of bentonite is quite high while the disposal of polymer can be readily conducted by adding agglomerator.

**6. Where should be the direction of gunning in shotcreting? (M4)**

During the construction of shotcrete, it is aimed at gunning the full thickness in one single operation and this helps to reduce the occurrence of possible delamination and formation of planes of weakness. Moreover, the nozzles should be held about 0.6m to 1.8m from the surface [2] and normal to the receiving surface. The reason of gunning perpendicular to the receiving surface is to avoid the possible rebound and rolling resulting from gunning at an angle deviated from the perpendicular. The rolled shotcrete creates a non-uniform surface which serves to trap overspray and shotcrete resulting from the rebounding action. This is undesirable because of the wastage of materials and the generation of uneven and rough surface.

**7. What is the mechanism in the formation of frost heave? (M5)**

In the past, it was believed that the formation of frost heave was related to the volumetric expansion of soil water which changed from liquid state to solid state. However, the increase of volume of changes in states for water at zero degree Celsius is only about 9% and the observed heaving is far more than this quantum.

In fact, the mechanism of frost heave is best explained by the formation of ice lenses [52]. In cold weather, ice lenses develop in the freezing zone in soils where there is an adequate supply of soil water. Soil particles are surrounded by a film of water which separates the soil particles from ice lenses. The moisture adhered to soil particles gets absorbed to the ice lenses on top of the soils and in turn water is obtained from other soil pores to replenish the loss of water to ice lenses. This process continues and results in pushing up of soils on top of the lenses and subsequently the formation of frost heave.

# *9. Module Eight: Tunneling*

## Objectives

| Element | Description | Objective No. |
|---|---|---|
| **Pipejacking** | | |
| Types of Pipejacking | Close mode, open mode and mixed mode | TP1 |
| | Pressure balance method/ compressed air method/ Slurry Shield TBMs | TP2 |
| Elements of Pipejacking | Packing materials in joints | EP1 |
| | Thrust wall | EP2 |
| | Grout holes | EP3 |
| | Intermediate jacks | EP4 |
| | Sharp-edges shield | EP5 |
| Problems of Pipejacking | Ground settlement | PP1 |
| | Changes in soil conditions | PP2 |
| **Pipe Ramming and Microtunneling** | | |
| Microtunneling | Definition | M1 |
| | Settlement | M2 |
| Pipe Ramming | Closed-end/open-end pipes | PR1 |
| | Loads | PR2 |
| **Tunneling with TBM** | | |
| TBM | Open shield and closed shield | TBM1 |
| | Single shield and double shield | TBM2 |
| | Slurry pressure | TBM3 |
| | Shotcrete | TBM4 |
| | Compensation grouting and normal grouting | TBM5 |
| Segmentally Lined Tunnels and Sprayed Concrete Linings (SCL) tunnels | Difference | SLT1 |
| | Grouting | SLT2 |
| | Segments | SLT3 |
| | Subdivision of tunneling faces | SLT4 |
| | | |

| **Element** | **Description** | **Objective No.** |
|---|---|---|
| **Design of Tunnels** | | |
| Design of Tunnels | Support approach and reinforcement approach | DT1 |
| | Ground-support interaction | DT2 |
| | New Austrian Tunneling Method, | DT3 |
| | “Cut-and-cover” method and “cover-and-cut” method | DT4 |
| | Ground freezing | DT5 |
| Design of Pipejacking | Speed of jacking | DP1 |
| | Stress concentration in pipe joints | DP2 |
| | Lubricants | DP3 |
| | Pipe joints | DP4 |
| | General | DP5 |

# Level One (Core FAQs)

## ***Part I: Pipejacking***

**1. What is the difference between close mode, open mode and mixed mode of trenchless methods? (TP1)**

(i) Close mode
It refers to mechanically operated TBM using bulkhead or slurry to balance earth pressure and groundwater. There is no manual access to the face of excavation.

(ii) Open mode
It refers free air hand-dug tunnel or compressed air handing tunneling with manual access to face of excavation.

(iii) Mixed mode
It is similar to close mode except that it allows access to the face of TBM for manual removal of obstructions.

**2. Which method of pipe jacking is better, pressure balance method or compressed air method? (TP2)**

Pressure balance method normally requires the use of mechanically operated tunnel-boring machine at its cutting head in pipe jacking. Slurry or steel bulkhead is commonly adopted to provide the balance of earth pressure and groundwater in front of the boring machine. Slurry used in balancing earth pressure and ground water pressure is constantly supplied to the face of the cutting wheel through slurry pipes. The excavated materials drop into a crusher for reduction in material size. Later, the debris and spoils will enter the spoil removal chamber near the invert of the shield and will be transported to ground level through slurry discharge pipes. This method of construction is normally adopted in sand and gravel. However, it suffers from the demerit that it is quite difficult to remove large rock boulders during the advancement of the machine. It is quite time-consuming for workers to go inside the relatively small airlock chamber and remove large bounders by hand tools.

The other type of pressure balance technique is called earth pressure balance method which is commonly used in clay and silty soils. It makes use of the principle of maintaining the pressure of excavation chamber the same as the pressure in ground. The excavated materials are transported through screw conveyor to the jacking pit.

Compressed air method in pipe jacking is commonly adopted in locations where groundwater table is high. An air pressure of less than 1 bar is usually maintained to provide the face support and to avoid water ingress. Pressurization and depressurization has to be conducted for workers entering and leaving the pipe-jacked tunnels. In case of porous ground, certain ground treatment like grouting has to be carried out. The removal of boulders by this method is convenient but it has the disadvantages of slow progress and significant noise problem generated by generators and compressors.

**3. What is the function of packing materials in the joint of concrete pipes in pipe jacking? (EP1)**

Packing materials are about 10mm to 20mm thick and are normally made of plywood, fibreboard or other materials. In case packing materials are absent in pipe joints for pipe jacking, then any deflection in the joints reduces the contact area of the concrete and it leads to spalling of joints due to high stresses induced. With the insertion of packing material inside the pipe joints, the allowable deflection without damaging the joint during the pipe jacking process can be increased.

**4. Is thrust wall an essential element in pipe jacking? Can it be omitted if there is insufficient depth for constructing normal thrust wall? (EP2)**

Thrust wall is an essential element in pipe jacking and it provides the reaction against the pipe jacking operation. In poor ground, consideration may be given to using piling or other methods to increase the stiffness of thrust wall. When there is insufficient depth to construct thrust wall (e.g. jacking through an embankment), a structure has still be constructed to provide the reaction to pipe jacking. In this case, the resistance to horizontal jacking loads is resisted by piles, ground anchors or other methods to reinforce the structure.

**5. Packers are normally introduced in pipe joints in pipe jacking. Why should packers be kept 20mm back from the edge of concrete? (EP1)**

Joint stress is induced in pipe joints during pipe jacking. Packers are normally installed in pipe joints to avoid localized stressing of joints leading to concrete crushing. In essence, packers should be elastic enough to take the reloading jacking force. Moreover, it should be thick enough to take the compression of maximum joint stress. Theoretically speaking, packers should be provided in all of the joint area except 20 mm back from the edge of concrete. The reason of such provision is to reduce the risk of local spalling of side edges.

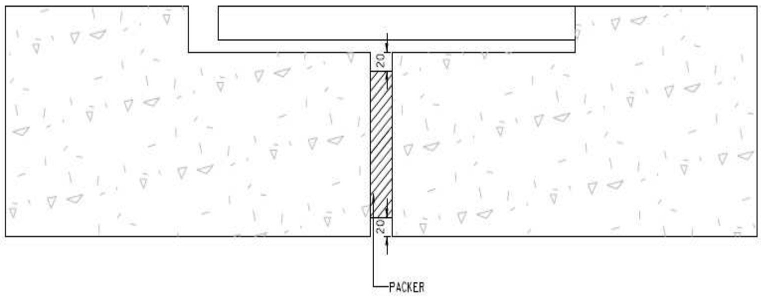



**Fig. Packer in pipe joint**

**6. In precast concrete jacking pipes, sometimes grout holes are designed inside these precast pipes. Why? (EP3)**

Grout holes are present in precast jacking pipes for the following reasons:

(i) They serve as the locations for injection of bentonite or other lubricant. Lubricant is used for both granular soils and cohesive soils to trim down the frictional resistance. For cohesive soils, the soils cannot get onto the pipes by the presence of lubricant and the shearing plane lies within the lubricant as suggested by R. N. Craig (1983). On the other hand, for granular soils, the lubricant mixes with soils with a significantly reduced friction. With the use of lubricant, longer pipe lengths can be jacked without the use of intermediate jacking station.
(ii) They provide the inlet locations for subsequent grouting works after completion of pipe jacking to fill completely the void space between the pipes and surrounding soils.
(iii) They are used as lifting holes when placing the precast jacking pipes into rails inside the jacking pits.

**7. Why are intermediate jacks designed in some pipe jacking projects? (EP4)**

When the process of pipe jacking stops, building up of resistance is very fast in some soil. For instance, increase in jacking force of 20%-40% is required for a stoppage of pipe jacking for just several hours. Hence, it is recommended that pipe jacking should be carried out in a continuous operation.

For a long pipeline, the frictional forces established between the jacking pipes and soil is high. Sometimes, such resisting forces may be so high that they can hardly be overcome by the jacks in jacking pits. Moreover, even if the jacks can overcome the high frictional forces induced during jacking, high loads are experienced in jacking pipes during driving. Jacking pipe's material e.g. concrete may not have sufficient strength to resist these stresses and hence pipe strength is another factor that governs the need for consideration of using intermediate jacks.

**8. Why does the problem ground settlement occur when pipe-jacking machine enters mixed ground with soils and boulders? (PP1)**

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.

**9. Why does pipejacking machine usually get stuck when the ground condition change from soil to very hard rock? (PP2)**

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.

**10. Would ground settlement occur ahead or behind the jacking face for pipe-jacking? (PP1)**

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.

**11. Why does ground heaving sometimes occur during pipe-jacking? (PP1)**

It is commonly recognized that ground settlement is one of the major concern in pipe jacking operation. However, engineer should also pay attention to the problem of ground heaving during grouting work of pipe-jacking. For instance, if excessive slurry or grout pressure is applied so as to exceed the overburden pressure, ground heaving would result. Alternatively, if the ground contains loose soils with high porosity, the same phenomenon also occurs. Proper control on the applied pressure and viscosity of grout/slurry is necessary to prevent such occurrence.

**12. What are the potential advantages of segmental tunneling when compared with hand-shield pipe-jacking?**

In segmental tunneling, the jacks are installed at the shield so that it is not necessary to install thrust wall at the jacking pit. This provides the opportunity for smaller size of the pit because of the absence of thrust wall. Moreover, as the jacking operation involves the jacking of small length of segmental liner plates and hence smaller force would be required for pipe jacking when compared with traditional pipe jacking (jacks at jacking pits) where the jacking force is needed to overcome long lengths of pipe drives. On the other hand, the use of segmental tunneling offers better control in alignment because the steering operation could be performed at the shield.

**13. How can Earth Pressure Balance TBM maintain stability of tunnel face? (TP2)**

Earth Pressure Balance (EPB) TBMs are used in excavating and advancing tunnels through any type of soft ground or soil condition, particularly below the water table. The EPB method consists of a cutting chamber located behind the cutterhead. This chamber is used to mix the soil with water foam. It is maintained under pressure by the mucking system. The ground at the cutting face is supported by earth pressure by balancing the advancement of the tunnel with the discharge rate of the excavated soil.

The underlying principle of the EPB method is that the excavated soil itself is used to provide continuous support to the tunnel face by balancing earth pressure against the forward pressure of the machine. The thrust forces generated from rear section of TBM is transferred to the earth in the cutterhead chamber so as to prevent uncontrolled intrusion of excavated materials into the chamber. When the shield advances at the face of excavation, the excavated soil is then mixed together with a special foam material which changes its viscosity or thickness and transforms it into a flowing material. This muck is then stored and is used to provide support and to balance pressure at the tunnel face during the excavation process.

**14. Why do Slurry Shield TBMs have difficulty when tunneling in clay? (TP2)**

In Slurry Shield TBMs, the slurry forms a filter cake on the face of excavation which has the following purposes:

(i) It provides the surface for slurry pressure to act on.
(ii) It acts as a seal against the y of groundwater into the tunnel.
(iii) In case the TBM breaks down, this filter cake serves as a sealing membrane at the tunnel face which allows man-entry into the excavation and working chamber upon provision go compressed air.

Slurry Shield TBMs are widely used for non-cohesive soils ranging from fine-grained sand to coarse-grained gravel. It is less suitable when operating in clay because:

(i) Most slurry separation plant could not separate clay from slurry. As such, the cost of frequent replacement of bentonite slurry is substantially increased.
(ii) Clayey materials to clog and cause blockages in slurry system leading to sudden pressure surges.

**15. Why are aqueous lubricants considered inappropriate for cohesive soils in pipe jacking?**

Lubrication performs effectively in pipe jacking by maintaining a layer of lubricant between the outside surface of jacking pipes and the adjacent soil. However, once the ground has collapsed onto the jacking pipe, the effect of lubrication would be greatly reduced. Hence, it is important to maintain sufficient pressure to avoid these occurrences.

Bentonite slurries are commonly used in silty, sandy and granular soils but it is not recommended to use in clayey soils because it may cause swelling in clays by absorption of water and this results in increased contact between the jacking pipe and soils. Hence, non-aqueous lubricants should be considered for cohesive soils.

**16. Swelling of clay develops significant stresses on pipe and its effect is even magnified by the use of lubricant. How can swelling inhibitors help to resolve this problem?**

Lubricants are introduced in pipe jacking to reduce the jacking loads. However, for pipe jacking in clay the swelling of clay is accelerated by water-based lubricant as the free water in the lubricant is readily absorbed by clay. To address the problem, swelling inhibitors are added to lubricants which enhance a reduction of free swelling for clays. In essence, the ability of swelling inhibitors to reduce free swelling of clay is achieved by the alternation of clay properties and formation of barrier on cavity surface.

**17. Why is sharp-edges shield normally used in pipe jacking with manual excavation? (EP5)**

The shield connected to lead pipe is normally sharp-edged in design because:

(i) It helps to reduce the resistance when the shield enters into soils;
(ii) It reduces the amount of soil dropping into the shield.

Sometimes the shield is equipped with jacks so that it allows tilting of the shield and adjustment could be made to the direction of pipe jacking.

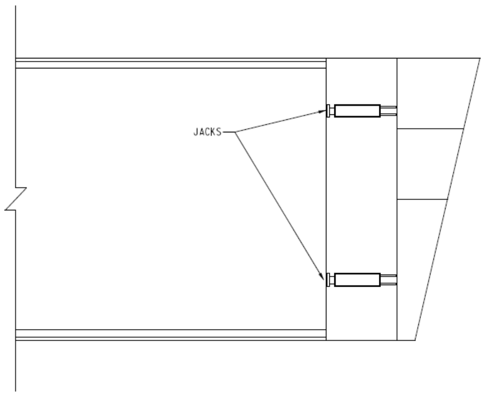



**Fig. Sharp-edged shield**

**18. In concrete pipe joint for pipe jacking, butt end type with mild steel band and socketed in-wall rubber ring type are commonly used. Which one is better?**

Butt end type with mild steel band is commonly used for stormwater application. The mild steel band serves to prevent lateral displacement if pipe joint during pipe jacking.

Socketed in-wall rubber ring type is normally used for sewer and pressure pipe situation and for pipe size exceeding 1200mm in diameter. The rubber ring is designed to provide a seal to pipe joint to ensure watertightness, which is essential in sewage pipelines.

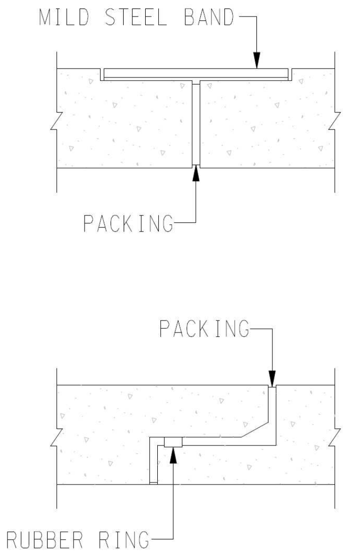


**Fig. Butt end type with mild steel band and socketed in-wall rubber ring type**

# Level One (Core FAQs)

## ***Part II: Pipe Ramming and Microtunneling***

### 1. What is the difference between pipe jacking and micro-tunneling? (M1)

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:

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

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

### 2. Does microtunneling refer to tunnel size less than 1,000mm? (M1)

The international definition of microtunneling includes pipe with diameter up to 1,000mm only. However, in the United States, it allows for larger pipe size when defining microtunneling in which pipe with diameters up to 144 inches are also counted within the ambit of microtunneling.

The basic concept of microtunneling has changed gradually owing to recent technological developments and past experience. For instance, the use of alignment control system with advanced surveying techniques allows for longer drives with good control and curved microtunneling. The use of automatic lubrication system enhances lower jacking forces. The employment of gripper in microtunneling machines helps develop adequate loads on cutter discs for cutting rock.

### 3. What are the situations which warrant the use of microtunneling instead of other trenchless methods?

There are two main advantages of microtunneling:

(i) Difficult ground conditions
Microtunnels could operate under a water head of 30m or more. It is capable of handling a wide range of soils such as cobble, boulders and rock without the need of dewatering.

(ii) Surface settlement
Surface settlement could be minimized by using microtunneling. For example, the use of earth pressure balance method in microtunneling helps balance the external soil loads and groundwater. Moreover, the rate of advancement of machine and the rate of excavation of tunnel face can be readily controlled so that it reduces the occurrence of over-excavation at tunnel face and hence the ground settlement.

**4. Why does settlement occur in microtunneling? (M2)**

Settlements occur in microtunneling, or other tunnel construction methods in two forms: large settlements and systematic settlements. The cause of large settlements is the over-excavation by microtunneling machine leading to the loss of stability at the tunnel face and the formation of empty space above the tunnel. The occurrence of large settlements is attributed to the improper operation of the tunneling machine or rapid unexpected changes in ground conditions.

Systematic settlements are mainly caused by the collapse of the radial overcut between the jacking pipe and the excavation. The annular space between the jacking pipe and the excavation is essential in microtunneling and pipe jacking for the following purposes:

(i) Reduction of jacking forces
(ii) Injection of the lubrication
(iii)Steering of the microtunneling boring machine

During tunneling, the soils may collapse onto the pipe, resulting in subsidence at the ground surface. Systematic settlements can be controlled by limiting the radial overcut and by filling the annulus with bentonite lubricant during tunneling, and with cement grout after tunneling is completed.

**5. In pipe jacking/microtunneling, it is commonly accepted that cover depths of jacking pipes cannot be too shallow (i.e. less than 2D where D is the diameter of jacking pipes). Why? (M2)**

For pipe jacking/microtunneling, the causes of large settlement are loss of face stability, failure to stabilize ground around shafts, presence of annular space around pipes and shield, drag along pipe joints, etc. The settlement mechanism of shallow depths of pipe jacking/microtunneling is the formation of a settlement trough on top of the jacking pipes. The width of the trough depends on soil properties; the larger is the cover depth of jacking pipes, the larger is the width of settlement trough. For the same soil volume loss due to pipe jacking/microtunneling, the width of settlement trough for shallow cover depth is smaller and therefore it results in a larger vertical maximum settlement.

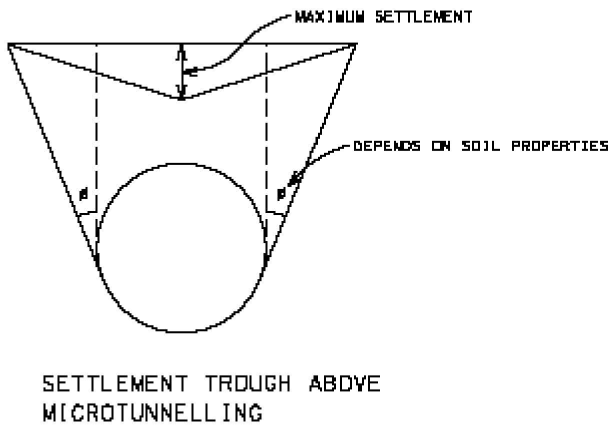



**Fig. Settlement trough in micotunneling**

**6. Should closed-end or open-end pipes be used in pipe ramming? (PR1)**

It is more common to adopt open-end pipes in pipe ramming because it is not readily to undergo surface heaving or pipe deflection when compared with closed-end pipes. Moreover, the use of open-end pipes requires lower ramming force.

Closed-end pipes are only used in the following conditions:

(i) Ground with poor self-support so that inflow of soils inside the pipes would render ground settlement and loss of support to utility

services.
(ii) Small size of pipes.

**7. Would pipe ramming increase the vertical soil loads on installed pipes? (PR2)**

Consider a certain cross section along the line of pipe ramming. When the pipe is rammed close to the cross section, the horizontal and vertical soil pressure would increase owing to the effect of soil compaction brought about by dynamic ramming operation. Upon reaching the cross section, soil pressure is redistributed around the pipe such that the vertical pressure above the pipes is reduced while the vertical pressure in pipe abutment locations is increased. When the pipe is advanced further, the load on pipes tend to increase owing to reorientation of soils around pipe wall. Finally, when the pipe is rammed some distance away from the cross section, a stable state is achieved in which there is smaller earth pressure on the pipe's top and higher vertical soil pressure on soils at both sides of the pipe.

**8. What are the situations which warrant the use of pipe ramming instead of other trenchless methods?**

There are two main distinct advantages of pipe ramming:

(i) Settlement and heaving of existing ground.
Where a pipe have to be installed under an existing railway or heavily-trafficked highway, it is almost impossible to install the pipes by open excavation. In particular, if the pipes to be installed are of shallow depth, the use of some trenchless methods (e.g. pipe jacking and heading) may cause considerable ground settlement because the soil loss within shallow zone would induce larger settlement. As such, the use of pipe ramming could resolve this concern. Pipe ramming is a displacement method which generally would not result in ground loss. For open-ended steel casing, the soils inside the pipe are not removed until the entire casing is installed in place.

(ii) Muck Disposal
For microtunneling, spoils are removed from the excavation face in a slurry so that the spoil are wet. Hence, sufficient space has to be provided to allow for drying of spoils or the wet spoils have to be removed off site immediately. For pipe ramming slurry is not used so that the spoil retains only it natural moisture content. Therefore, it is

easier to handle in-situ soils than wet spoils. On the other hand, the amount of spoil produced by pipe ramming is smaller when compared with pipe jacking and microtunneling.

# Level One (Core FAQs)

## ***Part III: Tunneling with TBM***

### 1. What is the difference between open shield and closed shield for TBM? (TBM1)

Open shield type TBM refers to those providing lateral support only. They can be further classified into single shield and double shield.

Closed shield type TBM refers to those providing lateral support and frontal support. Some common TBM method under this category includes compressed air TBM, slurry shield TBM, earth pressure balance machine and mixed confinement shield.

Compressed air TBM is suitable for cohesive soils under water table (e.g. ground with low permeability with no major discontinuities). Slurry shield TBM is suitable for soft ground and soft rock under water table and also for ground for high permeability. Earth pressure balance machine is suitable for soft ground and soft rock under water table. It is not recommended for very abrasive and hard ground.

### 2. What is the difference between single shield and double shield tunnel boring machine (TBM)? (TBM2)

In single shield TBM, it extends and moves forward by thrust cylinders on the last segment ring installed.

In double shield TBM, it consists of an extendable front shield which enhances the cutterhead to be extended. The gripper in the middle section of TBM is mobilized so that it pushes against the tunnel walls to react the boring forces. As these forces are dissipated, it allows the installation of lining segments during tunnel so that it increases the speed of tunneling. Upon completion of a trust stroke, the grippers are retracted and the end portion of TMB is pushed against the front shield by thrust cylinders.

Double shielded TBM is normally used in rock strata with geological fault zones and when a high rate of advancement is required. Single shielded TBM is more suitable to hard rock strata.

**3. In Slurry Shield TBMs, should high or low slurry pressure be maintained to support excavated face? (TBM3)**

In Slurry Shield TBMs, the slurry supports the wall of tunnel face in a manner similar to diaphragm wall. The bentonite forms a filter cake on the tunnel face on which the slurry exerts its pressure.

The face of a slurry shield tunnel boring machine is stabilized by bentonite slurry, which is kept under pressure. If the slurry pressures provided are too low, instability of the face results with occurrence of large settlements. On the contrary, in case the slurry pressures are kept too high, it leads to an excessive loss of bentonite and significant soil disturbance would occur.

**4. How can shotcrete stabilize tunnels? (TBM4)**

When shotcrete is sprayed on a rough ground surface, it fills small openings and cracks. It serves as initial support and also immediate support after excavation. It decreases the possibility of relative movement of rock bodies or soil particles and, therefore, controls the loosening of the exposed ground surrounding the tunnel.

Shotcrete lining could take up significant loads though it forms a flexible support system. In fact, shotcrete lining is expected to undergo large deformations which enable the intrinsic strength and self-supporting properties of the ground to be mobilized as well to re-distribute stresses between the lining and ground. During the deformation of lining, stresses within the shotcrete lining are relocated to the surrounding ground. As such, this mechanism of load transfer in turn generates subgrade reaction of the ground which gives support to the shotcrete lining.

Friction between shotcrete and the ground also reduces the differential movement of the ground. Even though shotcrete may not form a complete ring, the frictional forces between shotcrete and the ground could provide support to the ground.

**5. Can shotcrete be adopted as permanent linings in tunnels?** (TBM4)

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.

**6. What is the difference between compensation grouting and normal grouting? (TBM5)**

Compensation grouting is a technique to offset settlement induced during tunneling and underground excavation. The main idea of compensation grouting is to inject grouts into the zone between tunnel and overlying building to compensate for ground loss and stress relieve owing to underground excavation. The injection of grout changes the in-situ stress state and influences the soil deformation.

Compensation grouting could be implemented by fracture grouting and compaction grouting. Normal grouting can be carried out in situations such as post-grouting of mini-piles and filling of voids without the effect of compensation of ground loss and stress relieve.

**7. What are the differences between segmentally lined tunnels and Sprayed Concrete Linings (SCL) tunnels? (SLT1)**

The basic difference between segmentally lined tunnels and SCL tunnels is that the sprayed concrete in SCL tunnel is used for temporary linings only. Concrete linings in segmentally lined tunnels are designed with long-term load conditions with the adoption of appropriate safety factors. However, the dimension of linings in SCL tunnels is derived from a balance between safety of lining and cost consideration.

The quality of precast concrete linings can be better controlled in precasting yards. As such, the quality control of precast concrete lining is obviously better than that of sprayed concrete which depends on site workmanship [39].

**8. Why is grouting normally performed in segmental linings in tunnels? (SLT2)**

Grouting is usually carried out under pressure in segmental linings during tunnel construction because of the following reasons [62]:

(i) It helps in the uniform transfer of ground pressure to the linings.
(ii) The grout serves to reduce the surface settlement.
(iii) Grout fills up the annulus of tunnel linings so that it upholds the designed tunnel shapes.
(iv) The presence of grout aids in limiting groundwater seepage in the tunnels.

**9. What is the consideration in selecting the number of segments for segmental linings in tunneling? (SLT3)**

For the construction of tunnels by segmental linings, the choice of the number of segments affects the cost and durability of tunnels. With an increase in the number of segments, the number of joints also increases accordingly. This raises the potential for water ingress into the tunnels.

However, if the number of segments is kept to a minimum, the speed of the erection of segments can be increased. However, it is expected that higher bending moment would be induced in the tunnel rings for smaller number of segments and extra cost is incurred in the provision of additional reinforcement.

**10. What is the purpose of subdivision of tunneling faces in Sprayed Concrete Linings (SCL) method? (SLT4)**

In employing sprayed concrete lining methods for lining a large tunnel in soft ground, the tunneling face is normally divided into several parts because of the following reasons [62]:

(i) It enables early closure of part of the invert of the tunnel.
(ii) With the excavation in each part taking place at different times, it helps to reduce the area of exposure of the tunnel face so that there is better control on tunnel stability.
(iii) For unit advancement in any part of the tunnel excavation, the amount of excavation and sprayed concrete is reduced. As such, this allows for early provision of primary support.

**11. What are the functions of lattice girder in tunneling?**

Lattice girders are supporting elements in tunnels and they normally consist of steel bars laced together in triangular pattern. They are made to suit the shape of the tunnel. Owing to their small steel reinforcing area, they are not expected to contribute much to the overall support of tunnel. Instead, they are designed and provided for the following reasons:

(i) They have similar spacing with rock bolts and they are intended to provide temporary support to rock which is readily to loosen and fall.
(ii) Their presence provide an indication if sufficient thickness of shotcrete is applied.

# Level Two (Advanced FAQs)

## *Part I: Design of Tunnels*

**1. What is the difference in two common approaches in tunnel support, i.e. support approach and reinforcement approach? (DT1)**

For *support approach* it involves the application of reaction force at the face of excavation by using heavy structures, primarily ribs and lagging. For *reinforcement approach*, it involves the overall improvement of rock mass performance by techniques such as rock dowels, rock bolts and ground anchors. The target of reinforcement approach is to keep the rock and blocks from moving and loosening so that a large dead load of rock would not be exerted onto the support system. In fact, it holds the rock together and causes the ground around the opening to form a self-supporting ground arch around the opening.

There is a trend of tunneling industry to move from support approach to reinforcement approach because it requires less amount of structural steel support.

**2. What are the major factors affecting ground-support interaction in tunnels? (DT2)**

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.

### 3. Is NATM a tunneling method or a tunneling concept? (DT3)

In the original version of the New Austrian Tunneling Method, it is a tunneling concept with a set of main principles as follows:

(i) Application of thin-spayed concrete lining
(ii) Closure at invert to form a complete ring as soon as possible
(iii) Measurement of deformation until equilibrium is attained

It is a design concept in which the ground (soil or rock) enclosing the opening becomes a load-bearing element through formation of ring-like body. It uses all available means to develop maximum self-supporting capacity to provide support of the underground opening. In essence, it makes use of geological stress of surrounding soils to stabilize the opening. Therefore, NATM is a tunneling philosophy and concept rather than excavation techniques.

### 4. What is the difference between NATM and Sprayed Concrete Lining? (DT3)

The New Austrian Tunneling Method originated from rock tunnels and it requires the use of rockbolts and shotcrete swiftly after blasting. Institution of Civil Engineers has renamed it as Sprayed Concrete Lining for construction of tunneling linings by this method *in soft ground*.

There are some distinct differences in design philosophy between NATM and Sprayed Concrete Lining. In fact, it is not practical to mobilize inherent soil strength through deformation in soft ground. For lower strength of soils in soft ground, there should not be any delay in completing primary support and ground deformation should be reduced as much as possible.

### 5. Can pipe jacking be implemented in a fast manner? (DP1)

Energy is a function of load and speed. Damage may occur on the jacking pipes if it is incapable of absorbing the energy. Pipe jacking requires large energy in the process. Jacking pipes are low speed energy absorbers. To cater for the low-speed-energy absorbing characteristics of jacking pipes, hydraulic jacks should be designed to provide high loads with low speeds. Otherwise the excessive high speed generated by hydraulic jacks would cause kinetic damage to the pipes.

**6. In pipe jacking, can engineers roughly estimate the order of stress concentration in pipe joints? (DP2)**

Theoretical line and level of pipelines can hardly be achieved in pipe jacking. As such, the provision of angular deflection is made at pipe joints to accommodate such deviation. Normally, maximum allowable angular deflection at pipe joints in pipe jacking is $0.5^o$.

As a rule of thumb, the stress concentration at a pipe joint is about 3 times the joint stress resulting from uniform distribution of stress. As such, for grade 50MPa precast concrete pipe, the allowable uniform joint stress is expected to be one-third of its compressive strength, i.e. 16.67MPa.

**7. How can lubricants reduce the jacking forces? (DP3)**

The lubricating fluid serves to reduce the jacking force in the following ways:

(i) It saturates the overcut leading to partial and complete buoyancy of the jacking pipe in the cavity. As such, the contact surface area between the jacking pipe and soils is decreased.
(ii) It stabilizes the cavity by limiting the radial effective stress acting on the jacking pipe.
(iii) The interface friction angle between the soils and the jacking pipe are reduced by the lubricants.

**8. In the design of pipe jacking, what particular areas on pipe joints should engineers take care of? (DP4)**

Since in pipe jacking, the jacked pipes could hardly be jacked in the designed level and alignment and some deviation from the original one is commonly acceptable provided that the deviation are within the tolerance of the Contract. However, in order to avoid damage made to the pipe joints due to overstressing, it is necessary to estimate the stress concentrations resulting from these angular deflections.

Note: Pipe jacking is a trenchless method in which pipes are jacked underground from jacking pits and receiving pits.

**9. What are the differences in design between normal precast concrete pipes and pipes used for pipe jacking? (DP5)**

For pipes used for pipe jacking, they should possess the following characteristics:

(i) Pipes should have high concrete strength to withstand the stress induced during the jacking process.
(ii) There is tight tolerance in pipe dimension and the pipe joints are specially designed to provide trouble-free joint details. Two commonly available joints are rebated joint and butt end joint.
(iii) Pipes preferably should have smooth external concrete finishes to reduce the friction between the pipes and surrounding soil.

**10. How could ground freezing stop ingress of groundwater in excavation? (DT5)**

The concept of ground freezing involves the lowering of temperature of ground near the excavation area. Drillholes with designed spacing are installed so that a chilled brine or liquid nitrogen is introduced into the holes. Brine requires continuous circulation while liquid nitrogen is for rapid freezing and it is unrecoverable. With the addition of a chilled brine or liquid nitrogen, the groundwater is frozen into ice. Upon frozen, soils exhibit higher shear strength and the frozen zone acts as an impermeable barrier so that water could not enter the excavation zone.

**11. What is the difference between “cut-and-cover” method and “cover-and-cut” method in tunnel construction? (DT4)**

“Cut-and-cover” method involves the construction of open cut at the first place, followed by the construction of tunnels under open excavation. Upon completion of the structure, backfilling and reinstatement would be subsequently carried out.

“Cover-and-cut” method involves firstly the construction of cover followed by the second stage in which construction activities are carried out under the cover. As such, the disturbance to the public owing to constructional activities could be reduced to a minimum.

# 10. Module Nine: Site Investigation

## Objectives

| Element | Description | Objective No. |
|---|---|---|
| **Site Investigation** | | |
| Site Investigation | Dynamic probing and Standard penetration test | SI1 |
| | Cone penetration testing | SI2 |
| | Trial pit, trial trench and inspection pit | SI3 |
| | Single tube sampler, double tube sampler and triple tube sampler | SI4 |
| | Piston samplers | SI5 |
| | Standpipe, standpipe piezometer and piezometer | SI6 |
| | Drilling fluid in rotary drilling | SI7 |
| | Compressed air as drilling fluid | SI8 |
| | Vibrocoring | SI9 |
| **Testing** | | |
| Triaxial Test | Consolidation pressures | TT1 |
| | Consolidated undrained test/consolidated drained test | TT2 |
| | Multistage | TT3 |
| Packer Test | Multiple packer test/single packer test | PT1 |
| Sedimentation Analysis | Hydrogen peroxide | SA1 |
| Rowe cell and Oedometer apparatus | Difference | RC1 |

# Level One (Core FAQs)

## ***Part I: Site Investigation***

### 1. What is the difference between Dynamic Probing and Standard Penetration test? (SI1)

Standard Penetration Test (SPT) is an in-situ dynamic penetration test to provide information on the properties of soil. It may also collect a disturbed soil sample for grain-size analysis and soil classification. SPT involves the driving of a standard sampler through a distance of 450mm into the bottom of a borehole using the standard weight of 63.5kg falling through 760mm.

Dynamic Probing Test as per BS1377: Part 9: 1990 involves the driving of a metal cone into the ground through a series of 1-metre length steel rods. These rods are driven from the surface by the hammer system on the rig which drops 63.5 kg weight onto the rods through a fall of 760mm. The number of blows that is required to drive the cone down each 100mm increment is then recorded until a required depth is reached or a refusal is achieved. Dynamic Probing has many applications. For instance, it may be used to estimate the depths of at the interface between hard and soft strata and to trace the outline of objects buried underground.

SPT test is used to provide valuable information on soil properties. However, the main use of dynamic probing is *to interpolate information* between boreholes/trial pits swiftly and to supplement information found from boreholes and trial pits at a low cost. For instance, dynamic probing is carried out close to a borehole where the underground conditions are identified. As such, by using the dynamic probing, the result of borehole can be extended to other areas in between two boreholes.

### 2. When would engineers use Cone Penetration Testing instead of Standard Penetration Test? (SI2)

(I) Standard Penetration Test is carried out in boreholes at 1.5-2m intervals. However, Cone Penetration Testing allows a continuous record of ground resistance profile.
(II) Disturbance to ground is less by Cone Penetration Testing when compared with Standard Penetration Test.
(III) The use of Cone Penetration Testing is faster and cheaper when compared with the combination of boring, sampling and Standard Penetration Test.

**3. How to identify sand and clay from the results of Cone Penetration Testing? (SI2)**

Cone Penetration Testing measures the pressure at the end of cone (end resistance), friction on sleeve and pore water pressure. Friction ratio is defined as the ratio of friction/end resistance.

For clay, typical CPT results exhibit low end resistance, high friction ratio and high pore water pressure.

For sand, typical CPT results exhibit high end resistance, low friction ratio and low pore water pressure.

**4. What is the application of Continuous Piezocone Penetration Test? (SI2)**

Continuous piezocone penetration test basically consists of standard cone penetration test and a measurement of pore water pressure. Three main parameters, namely sleeve friction, tip resistance and pore water pressure measurement are measured under this test.

Pore water pressure generated in the soils during penetration of the cone is measured. An electrical transducer located inside the piezocone behind saturated filter is used for the measurement. By analyzing the results of pore pressure with depth, the stratigraphy of fine-gained soils with different layers is obtained readily.

**5. What is the difference between trial pit, trial trench and inspection pit? (SI3)**

A trial pit is used for obtaining information on the subsurface soil conditions. It allows logging of the various soils types and soil sampling. Typical size of trial pits has minimum base plan area of 1.5m$^2$. Trial trench serves the same purpose of trial pit except that they differ in size and dimension. For instance, the length of a trial trench is normally larger than its width by a certain factor (e.g. 5) to cater for its “trench” shape.

An inspection pit is a pit used for identifying and positioning of underground utilities and structure.

## 6. How can trial pits be made in slopes? (SI3)

In slopes, trial pits are transformed into another method called slope surface stripping. It involves the removal of slope surface protection and vegetation to reveal the soil conditions below slope surface for inspection purpose. The strip is about 0.3m to 0.5m wide and 150mm to 300mm deep and it usually extends from the crest to the toe of slopes. Slope surface stripping is commonly used in Hong Kong.

## 7. What are the differences among single tube sampler, double tube sampler and triple tube sampler? (SI4)

Core barrel samplers are originally designed to sample rock. In single tube sampler, the core barrel of the sampler rotates and this poses the possibility of disturbing the sample by shearing the sample along certain weak planes. Moreover, the cored samples are subjected to erosion and disturbance by the drilling fluid.

For double tube samplers, the tube samplers do not rotate with the core barrels and the samplers are not protected against the drilling fluid. The logging of samples presents difficulty for highly fractured rock. The triple core barrel basically consists of a double core barrel sampler including an addition of a stationary liner which is intended to protect the cored samples during extraction. Therefore the quality sample obtained from triple core barrel is the best among the three types of barrels mentioned above.

## 8. How do Piston samplers function? (SI5)

In sampling clays or silts, Piston sampler is lowered into boreholes and the piston is locked at the bottom of the sampler. This prevents debris from entering the tube prior to sampling. After reaching the sampling depth, the piston is unlocked so that the piston stays on top of the sample going into the tube. Prior to the withdrawal of the sampler, the piston is locked to prevent the downward movement and the vacuum generated during the movement of the piston from the sampler’s end aids in retaining the samples recovered. As such, sample recovery is increased by using Piston samplers.

## 9. What is the difference between standpipe, standpipe piezometer and piezometer? (SI6)

A standpipe normally contains plastic pipes with perforated holes at the base. The annular space between the perforated tube and casing is filled with gravel or sand backfill. Under such an arrangement, standpipe is used to measure water level of a certain region.

A standpipe piezometer is a type of piezometer which measures pore water pressure at a certain level. It consists of plastic pipes without holes. The tip of the standpipe piezometer is perforated and the annular space between the tip of the piezometer and soil is filled with sand while the annular space between other parts of plastic tube and soil is filled with cement/bentonite grout to seal off water from entering the region of piezometer tip. This enables the pore water pressure in the region of piezometer tip to be measured. In essence, standpipe piezometers are installed to study the pore water pressure of a specified depth below ground. However, it suffers from the disadvantage that the response time is relatively slow in clayey soils. Reference is made to Marius Tremblay (1989).

**10. What is the function of drilling fluid in rotary drilling in site investigation? (SI7)**

Drilling fluid in rotary serves two main purposes:
(i) Facilitate the rotation of drilling tube during rotary drilling;
(ii) Act as a cooling agent to cool down heat generated during drilling operation.

Traditionally, water is normally employed as drilling fluid. However, it suffers from the following drawbacks:
(i) It affects the stability of nearby ground with the introduction of water into the borehole (borehole for soil; drillhole for rock);
(ii) It affects the quality of sample by changing the water content of soil samples collected from the borehole/drillhole.

Substitutes are available in market to replace water as drilling fluid (e.g. white foam).

**11. What are the reasons of using compressed air as drilling fluid? (SI8)**

For rotary drilling in ground investigation works, drilling fluid is normally used to clear and clean the cuttings from the drilling bits and transport them to the ground surface. Moreover, it also serves to produce a cooling effect to the drilling bit. In addition, the stability of boreholes can be enhanced and the drilling fluid also produces lubricating effect to the bits.

Compressed air when used as a drilling fluid possesses several advantages. Firstly, the use of compressed air can reduce the loss of fluid during circulation which is commonly encountered for water being used as drilling fluid. Secondly, the efficiency of air to clean drilling bits is higher than other types of drilling fluids. Thirdly, the moisture condition of in-situ soils would not be affected by air when compared with water as drilling fluid. In addition, in cold countries the occurrence of freezing of drilling water/mud can be avoided by using air. However, special attention should be taken to avoid breathing the generated dust when compressed air is employed as drilling fluid and dust suppression measures have to be properly implemented.

**12. Why is vibrocoring frequently used in marine ground investigation? (SI9)**

If only shallow marine ground geotechnical information is required for design purpose, vibrocoring is inevitably a good choice for sampling disturbed samples. In vibrocoring, a core barrel and an inner liner usually of 100mm diameter and 6m long are vibrated into the seabed. Since the installation of vibrocoring involves the vibration of barrels, there is considerable disturbance of recovered samples. Vibrocoring has the merit of the fast speed of sample recovery (e.g. up to 14 cores can be obtained in one day). Moreover, the cost of vibrocoring operation is low when compared with other viable marine geotechnical investigation options.

**13. 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.

# Level Two (Advanced FAQs)

## *Part I: Testing*

**1. In conducting triaxial test to determine shear strengths for soil samples, what consolidation pressures should be specified? (TT1)**

It appears that the selection of consolidation pressure is independent of in-situ soil stress theoretically. However, this may not be correct because the actual shear strength envelopes for soils are non-linear over a wide range of stresses. Therefore, consolidation pressure corresponding to the range of stresses relevant to site condition should be adopted.

**2. In determining the effective stress parameters of a soil sample, which test is preferable, consolidated undrained test or consolidated drained test? (TT2)**

The effective stress parameters of a soil sample can be obtained from both consolidated undrained test and consolidated drained test. However, consolidated undrained test is normally selected because of the following reasons:

(i) Time taken for consolidated undrained test is shorter than that of consolidated drained test. It is because consolidated drained test requires the full dissipation of excess pore water pressure of the soil during testing and it takes long time when soils of low permeability are tested.
(ii) Useful information can be obtained from the stress path of consolidated undrained test.
(iii) Failure occurs in lower stress level when compared with consolidated drained test.

**3. Why is shear box test not a better alternative to triaxial test in determining shear strengths of soils? (TT2)**

The test procedure is simpler for shear box test. However, it suffers from the demerit that drainage conditions are not easily controlled and pore water pressure cannot be measured. Moreover, the plane of failure is governed by the test itself rather than the properties of soil. It is likely that shear stress distribution across the soil sample is not uniform. The above limitations may affect the accuracy and reliability of test results.

One of the advantages of shear box is that the test could be continued to large strains so that residue shear strength could be determined. In fact, triaxial test has mostly replaced shear box test for normal application.

**4. Multistage triaxial test may not be preferable for consolidated drained test. Why? (TT3)**

In multistage consolidated drained test, the soil sample is consolidated under all round pressure and then loaded by applying an axial stress. Prior to failure, loading is stopped and the specimen is consolidated under a higher confining pressure. The above steps are repeated for 3 stages to obtain the failure envelope.

The main problem associated with multistage consolidated drained test lies in the practical difficulty in determining the failure state of the soil sample. Judgment has to be made regarding the condition of “immediately prior to failure” on stress strain curves. It is not uncommon that wrong estimation of the failure state occurs when interpreting the stress strain curves. When there is an underestimation of deviator stress at failure, it would result in overestimation of friction angle and underestimation of cohesion. In case actual failure of soil samples occurs before visual recognition, the sample undergoes overstressing so that the deviator stress at failure in later stages is reduced. As such, this leads to overestimation of cohesion and underestimation of friction angle.

**5. What are the limitations in multi-stage test of triaxial tests? (TT3)**

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.

**6. Why is multiple packer test instead of single packer test sometimes adopted in testing permeability of rock? (PT1)**

Packer test is used in unlined drillholes in rock to test the permeability. In single packer test, the hole is drilled to the bottom of first test section and the top of the test section is sealed off by a packer. Water is then delivered to the test section and it is kept at constant pressure and the flow is measured.

In highly fractured rock there is a high chance that water tends to leak around the packer which gives inaccurate result. As such, multiple packers are adopted instead in which three sections of the drillhole are sealed up and water is pumped to them at equal pressure. This eliminates the tendency for water to flow around the packers from the middle section. Hence, a more accurate result could be obtained by measuring flow from the middle section alone.

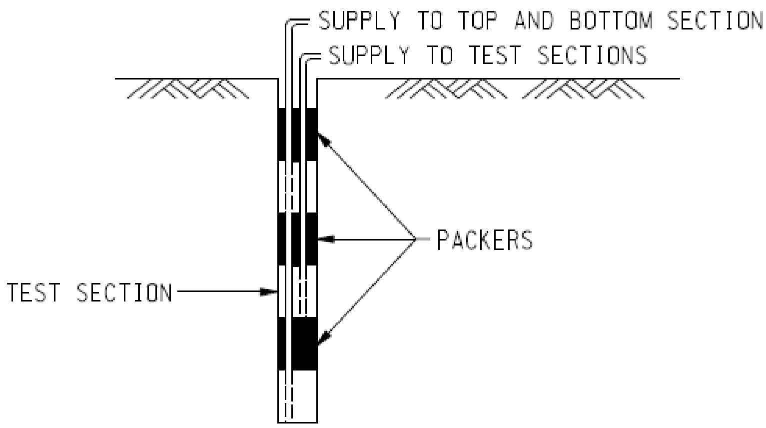



**Fig. Illustration of multiple pack**

**7. What is the purpose of adding hydrogen peroxide in sedimentation analysis? (SA1)**

There are two major techniques of particle size distribution:

(i) Sieve analysis – for soil particles larger than 60 $\mu$m they can be separated by this method.
(ii) Sedimentation analysis – for soil particles smaller than 60 $\mu$m, they are too small to be sieved by sieve analysis. Instead, the particle size distribution is worked out from the rate of settlement of soil particles suspended in water by Stoke's law.

In sedimentation analysis, the soil under testing is firstly boiled with little distilled water to wet and break up the particles. After that, hydrogen peroxide is added *to remove any organic material*. Then the whole mixture is allowed to stand still for a night and then boiled again to remove hydrogen peroxide.

**8. Is it worthwhile to carry out tests on particle density of soil particles for geotechnical design?**

Particle density of soils is defined by the ratio of soil particle mass and soil particle volume. Depending on soil types, the range of variation of soil particle density varies not significantly, i.e. by 4%. Therefore, it may not be worthwhile to order laboratory tests and incur additional expenditure just to determine the particles density by recognizing that the variation of particles density is not significant.

**9. How do we compare Rowe cell and Oedometer apparatus? (RC1)**

The advantages of using Rowe cell over oedometer apparatus are:

(i) It possesses the control facilities for drainage and for the measurement of pore water pressure.
(ii) It is capable of testing larger diameter soil samples. Hence, more reliable data can be provided by using Rowe's cell because of the relatively smaller effect of structural viscosity in larger specimens.
(iii) Rowe cell uses hydraulic loading system which is less susceptible to the effect of vibration than oedometer apparatus.

# *11. Module Ten: Waterworks*

## Objectives

| Element | Description | Objective No. |
|---|---|---|
| **Pipelines** | | |
| Pipelines | Difference between ductile iron pipes and steel pipes | P1 |
| | Asbestos cement | P2 |
| Ductile Iron Pipes | Cement-mortar lined | DIP1 |
| | Polyethylene encasement | DIP2 |
| Design of Watermains | Air | DW1 |
| | Double air valves and single air valves | DW2 |
| | Gate valve | DW3 |
| | Puddle flange | DW4 |
| | Butterfly valves | DW5 |
| | Bolt tightening sequence | DW6 |
| | Swabbing | DW7 |
| | Air chamber and surge tank | DW8 |
| **Thrust Blocks** | | |
| Design of Thrust Blocks | Hydrodynamic forces | DTB1 |
| | Unbalanced force in horizontal bends | DTB2 |
| | Restrained joints | DTB3 |
| | Reinforcement | DTB4 |
| **Water Retaining Structures and Reservoirs** | | |
| Design of Water Retaining Structures | Allowable crack width | DWRS1 |
| | Indirect tensile strength | DWRS2 |
| | Critical steel ratio | DWRS3 |
| | Movement joints | DWRS4 |
| | Mild steel or high yield steel | DWRS5 |
| | Waterstop | DWRS6 |
| | Watertightness test | DWRS7 |
| **Pumps and Pumping Station** | | |
| Pumps | In series and in parallel | P1 |
| | Radial flow pumps | P2 |

| **Element** | **Description** | **Objective No.** |
|---|---|---|
| | Axial flow pumps | P3 |
| | Backward curved vanes and forward curved vanes | P4 |
| | Best efficiency point and operating point | P5 |
| | Minimum volume of sump volume | P6 |
| | Screw pumps | P7 |
| Pumping Station | Waterproofing system | PS1 |
| | Cement plaster, cement render and cement screed | PS2 |
| | Corbel beams | PS3 |
| | Lightweight infilling material in raft foundation | PS4 |

# Level One (Core FAQs)

## ***Part I: Pipelines***

**1. Which one is better, ductile iron pipes and mild steel pipes as pressurized pipelines? (P1)**

For watermain pipe size less than 600mm, ductile iron is normally used because internal welding for steel pipes below 600mm is difficult to be carried out. Moreover, it requires only simple jointing details which allows for a faster rate of construction. For watermain pipe size above 600mm, steel pipes are recommended because steel pipes are lighter than ductile iron pipes for the same material strength and therefore the cost of steel pipes is normally less than that of ductile iron pipes. In addition, in areas of difficult access the use of lighter mild steel pipes has an advantage over ductile iron pipes for easy handling.

**2. What is the difference between ductile iron pipes and steel pipes in resisting external loads? (P1)**

Ductile iron pipes normally possess thicker pipe walls and are generally stiffer than steel pipes. As such, it replies less on side fill soils to support external loads. Hence, it is not necessary to achieve highly-compacted soils for ductile iron pipes for sustaining external loads.

For steel pipes, owing to less stiffness associated with thinner pipe walls, it relies heavily on the stiffness of backfill soils in resisting external loads. Hence, to enhance the external load-carrying capacity of steel pipes, the most convenient methods are to improve the quality of backfill materials and to increase the level of soil compaction.

**3. What does the pipe thickness of ductile iron pipes generally larger than that of steel pipes? (P1)**

Both steel pipes and ductile iron pipes use hoop stress equation to model internal pressure design. The difference in pipe thickness arises as a result of more conservative approach in DI pipes.

For ductile iron pipes, surge pressure is considered as part of design pressure and they are added together before applying a safety factor of 2 as follows:

$$t = \frac{F(P+S)D}{2Y}$$

where t = Pipe thickness
F = Factor of Safety of 2
P = Working pressure
S = Surge pressure
Y = Yield strength of ductile iron

For steel pipes the design of working pressure is based on 50% of steel yield strength (i.e. a factor of safety of 2). The presence of surges could be allowed to increase the stress in pipe to 75% of yield strength. The design is based on the following steps:

(A) If surge pressure is less than or equal to one-half of working pressure, the pipe shall be designed using working pressure only with 50% yield strength as allowable stress.

$$t = \frac{PD}{2Y}$$ where Y = 50% of yield strength

(B) If surge pressure is more than or equal to one-half of working pressure, the pipe shall be designed using working pressure and surge pressure only with 75% yield strength as allowable stress.

$$t = \frac{(P+S)D}{2Y}$$ where Y = 75% of yield strength

For case A**,** the use of 50% yield strength is essentially the same of adopting a safety factor of 2 in DI pipe design. However, as surge pressure is not considered, the thickness calculated is smaller than that in DI pipe design.

For case B, the use of 75% yield strength is essentially the same of adopting a safety factor of 1.33 in DI pipe design. As such, the thickness calculated is smaller than that in DI pipe design.

**4. In the design of watermain, the normal practice is to use ductile iron for pipe size less than 600mm and to use steel for pipe size more than 600mm. Why? (P1)**

For watermain pipe size less than 600mm, ductile iron is normally used because internal welding for steel pipes below 600mm is difficult to be carried out. Moreover, it requires only simple jointing details which allow for faster rate of construction. For watermain pipe size above 600mm, steel pipes are recommended because steel pipes are lighter than ductile iron pipes for the same material strength and therefore the cost of steel pipes is less than that of ductile iron pipes. In addition, in areas of difficult access the lighter mild steel pipes pose an advantage over ductile iron pipes for easy handling.

**5. Why are some ductile iron pipes cement-mortar lined? (DIP1)**

Cement mortar lining provides a high pH in the inner surface of ductile iron pipes and it serves as a physical barrier to water to guard against corrosion of iron from acidic water. Moreover, owing to smooth nature of cement lining, it results in the provision of high flow.

For cement-mortar lined ductile iron pipes to convey water, water infiltrates the pores of lining and releases some calcium hydrate. The freed calcium hydrate form calcium carbonate with calcium bicarbonate in water so that it serves to clog the pores of linings and avoid further permeation of water. Moreover, iron also reacts with lime to precipitate iron hydroxide which also seals the pores of linings. As such, the lining provides both chemical and physical barrier to aggressive water.

**6. In some countries like the United States, asphaltic seal-coat is used in cement mortar lining of ductile iron pipes. Why? (DIP1)**

The original intention of adding a thin asphaltic seal-coat on freshly placed cement-mortar lining is to reduce water loss during hydration so as to achieve better curing of the linings. In fact, it also helps hinder the leaching of cement by corrosive water. Otherwise, leachates from cement linings may cause a rise in pH in water.

However, asphaltic seal-coat is considered undesirable from environmental point of view. The seal-coat material is solvent-based which contains volatile organic compounds, which is an air pollutant.

**7. What is the purpose of polyethylene encasement for ductile iron pipes? (DIP2)**

The provision of polyethylene encasement avoids the direct contact of ductile iron pipes with surrounding aggressive environment. However, polyethylene encasement is not intended as a water-tight system. Initially moisture between ductile iron pipes and polyethylene encasement contains some oxygen which shall be ultimately depleted and this would put a stop to the oxidation reaction. This process leads to a stable and stagnant environment near the pipe and it provides a protective environment.

**8. Why is the presence (or absence) of air undesirable in pressurized pipelines? (DW1)**

Air valves are broadly classified into two main types: single air valves and double air valves. Single air valve contains a small orifice air valve which allows automatic release of a small amount of accumulated air during normal operation of the pressurized pipeline. Double air valves contain a small orifice air valve and a large orifice air valve. The large orifice air valve exhausts air automatically during filling and permits admission of air during emptying of the pipeline. However, it cannot perform the function of a small orifice air valves.

The presence of air in the pressurized pipeline is undesirable due to the following reasons:

(v) The presence of air causes significant impedance to water flow and in the worst case it may even cause complete blockage of the system.
(vi) The air induces considerable head loss to the system and causes the wastage of useful energy.
(vii) It may cause serious damage to meters and even cause inaccurate reading of the meters.
(viii) The presence of air causes water hammer damage to the pipeline.

The absence of air (i.e. during emptying operation of pipeline in routine maintenance) may also generate problems owing to the following:

(i) The suction generated draws in dirt and mud through faulty connections and cracks in pipelines.
(ii) The seals, gaskets and internal accessories will be suctioned inside the pipelines.
(iii) Sometimes, the suction forces may be so significant to cause collapse of pipelines.

One may query that if a large orifice air valves can perform the functions of filling and release of air, why is it necessary to add small orifice air valves in the pipeline system for release of accumulated air during normal operation? The reason is that the air accumulated at the high points of pressurized system will be expelled through the large orifice air valves (in case no small orifice air valves are installed in the system) upon starting of a pump and with such rapid outflow of air through the large valves, high slam pressure may be produced resulting in the damage of the pipelines.

**9. In the design of watermains, how to decide the usage of double air valves and single air valves? (DW2)**

Single air valves allow squeezing air out of the pipeline in automatic mode in high-pressure condition and are normally designed in high points of watermain in which air voids are present. Double air valves basically serve the same purpose except that it has another important function: it can get air into/out of the pipeline during low-pressure condition.

In WSD practice, watermain are normally divided into sections by installation of sectional valves to facilitate maintenance. In a single isolated pipeline section bounded by two sectional valves, at least a double air valve should be installed. During normal maintenance operation like cleansing of watermain, water inside pipelines is drawn from washout valves. However, as normal watermain is subject to very high pressure like 1.5MPa and the sudden withdrawn of water will cause a transient vacuum condition and will damage the watermain. Therefore at least one double air valve should be present to allow air to squeeze in to balance the pressure and this protects the pipeline from damaging.

In essence, for local high points single air valves should be installed. Within a section of pipeline, at least one double air valve should be installed.

**10. Why are two gate valves required in normal practice to form a washout valve?**

In fact, the situation is analogous to that of fire hydrants in which two gates valves are installed with a single fire hydrant. Washout valves are used for normal maintenance work of watermain like allowing flowing out of water during cleaning of watermain. At the junction where a tee-branch out to a washout point, a gate valve is installed to separate the two pipelines. However, this gate valve is open during normal operation while another gate valve further downstream is installed (closed during normal operation).

If the downstream gate valve is not installed in position, then the pipe section of branched-out watermain will be left dry during normal operation and there is a high probability that damage to watermain and frequent leakage would occur. With the downstream gate valve installed, the segment of branched-out watermain contains water in normal operation. In case there is any leakage, it can be readily detected by using the two gate valves.

**11. What is the purpose of embedding puddle flange inside the walls of closed valve chambers? (DW4)**

When valves are closed to stop water flow in a pipeline, a thrust is generated along the direction of the pipeline. Hence, it is necessary to restrain the valves during closure to prevent it from moving in the thrust direction. If the closed valve is situated inside valve chambers, it is connected to a puddle flange embedded inside a wall of the chamber. As such, the closed valves can be effectively restrained from the thrust action during the closure of valves.

**12. What is the difference between "linear characteristic" and "equal percentage characteristic" in controlling butterfly valves? (DW5)**

For "linear characteristic", the flow rate is directly proportional to the amount of travel of butterfly disk. For example, at 25% open of butterfly disk, the flow rate of 25% of maximum flow.

For "equal percentage characteristic", equal increment of opening of butterfly disk leads to equal percentage change in flow rate. For example, when butterfly disc open from 30% to 40%, it generates a change in flow rate of 50%. Therefore, when butterfly disc open from 40% to 50%, it also generates a change in flow rate of 50%. If the flow rate at 30% is 200$m^3$/s, then the flow rate of 40% and 50% open are 300$m^3$/s and 450$m^3$/s respectively.

The use of different characteristics depends on the amount of pressure drop available to the butterfly value. Should more than 25% of system pressure drop is available to the butterfly value, then the employment of linear characteristic would provide the best results. On the contrary, if less than 25% of system pressure drop is available to the butterfly value, then the employment of equal percentage characteristic would be a better choice.

## 13. What is the purpose of bolt tightening sequence in flanged joints? (DW6)

Bolted flange joint is widely used in watermains. In essence, a bolt axial tension is applied by the torque control method. The preload values are recommended by gasket suppliers to control gasket crushing and to achieve proper gasket seating stress. A proper bolt tightening sequence in flanged joints is essential to control stress variation in flange joint components. Otherwise, leakage occurs at flanged pipe joints during operating conditions.

Most joint surface of joints is not completely flat. The sequence of bolt tightening exerts a huge effect on the resulting preloads. Since joints containing gaskets have a comparatively low compressive stiffness, bolt preloads in such joints are particularly sensitive to the tightening sequence. Owing to the compression of joint surfaces, tightening one bolt close to another pre-loaded bolt will affect the preload generated by the firstly-tightened bolt. A proper bolt tightening sequence ensures that an even preload distribution is achieved in the flanged joint.

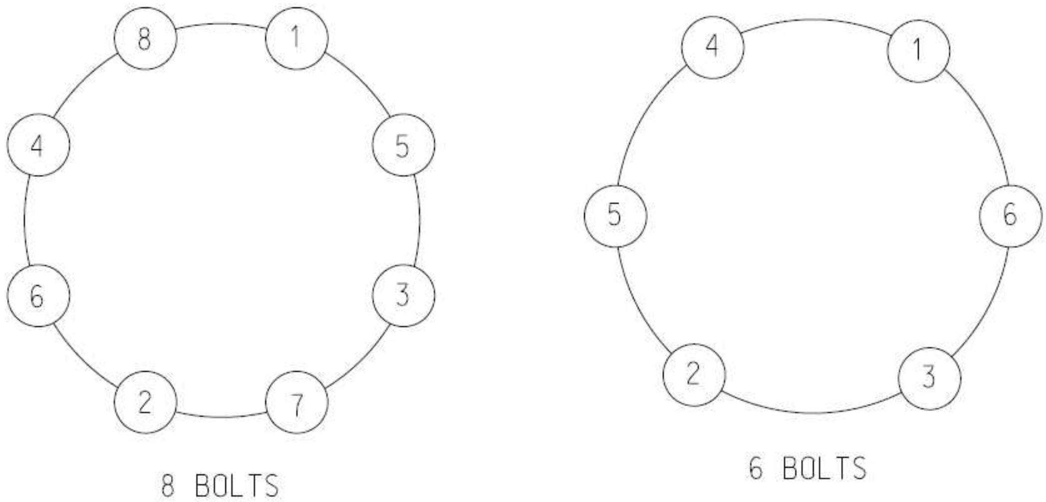



**Fig. Bolt tightening sequence in flanged joints**

## 14. What is the reason of retightening in flanged joints? (DW6)

Traditional gaskets are non-elastic in nature. As such, this property of gasket results in a reduction in the bolt's preload as time goes by. In fact, this phenomenon usually takes place shortly after installation leading to bolt relaxation. To reduce the effect of bolt relaxation which may cause subsequent water leakage during operation, retightening the bolts is carried out some time latter after initial tightening.

**15. After the construction of watermain, prior to hydrostatic pressure test, swabbing is carried out. What is the purpose of swabbing? (DW7)**

Pipelines should be tested before commissioning to check the strength of watermain and the absence of leak. Before carrying out hydrostatic pressure test, swabbing is conducted to clear out rubbish and dirt left inside the pipeline during construction. Swabbing is required for pipes less than 600mm diameter because for larger size of pipes, they can be inspected internally to ensure cleanliness.

After carrying out of hydrostatic pressure test, test for water sterilization is then conducted which involves collecting water sample from the pipeline. The purpose is to check the water quality like colour, turbidity, odor, pH value, conductivity etc. and is compared with the quality of water drawn from water supply point.

**16. What are the controversial health issues of using asbestos cement pipes for watermain? (P2)**

There are suspected health hazard for using asbestos cement pipes for watermain. In drinking water the gastrointestinal tract cancer risk depends on the amount of asbestos swallowed. When asbestos cement pipes are in good condition, there should be little safety problem. However, when the pipes become aged so that some may break down, it then become a great hazard. When someone takes a little asbestos which are distributed to other parts of the body so that no single parts of the body have excessive amount of asbestos, the risk should be theoretically on the low side. The harmful effect of asbestos is its ability to accumulate in human body. The microscopic fibers lodged in tissues can act like time bombs and cause cancer years later. Since asbestos exposure is cumulative, adults have three or four decades to develop cancer after exposure while youngsters have six or seven.

However, according to the findings of WHO, asbestos fibres are too large to be absorbed during the digestion. Therefore, the chance of significant transmission of asbestos fibres would seem to be low. Some evidence suggests that high density asbestos-cement products pose no detectable risk to the public because asbestos fibres are carcinogenic only when inhaled but not ingested.

**17. In road opening, it is sometimes noted that asbestos cement pipes are broken up into pieces. Why? (P2)**

Asbestos cement is mainly a mixture of cement and asbestos fibres with density greater than 1000kg/m$^3$. It contains about 10% asbestos fibres and it is a light grey hard material. The fibres are tightly held in cement mixture and they shall be discharged if asbestos cement undergoes significant disturbance such as drilling and sawing.

However, one of the important characteristics of asbestos cement is the relative brittleness. As such, asbestos cement pipes can be broken easily when falling at height or driven over by heavy vehicles. Hence, it is not uncommon that asbestos cement pipes are observed to break up into pieces in inspection pits.

**18. What is the difference between air chamber and surge tank in pressurized pipelines? (DW8)**

Air chambers and surge tanks are normally installed in watermain to ease the stress on the system when valves or pumps suddenly start up and shut down. A surge tank is a chamber containing fluid which is in direct contact with the atmosphere. For positive surge, the tank can store excess water, thus preventing the water pipes from expansion and the water from compression. In case of downsurge, the surge tank can supply fluid to prevent the formation of vapour column separation. However, if the relief of surge pressure is significant, the height of surge tank has to be large and sometimes it is not cost-effective to build such a large tank. On the contrary, an air chamber can be adopted in this case because air chamber is an enclosed chamber with pressurized gases inside. The pressure head of the gas inside the air chamber can combat the hydraulic transient. The volume of liquid inside the air chamber should be adequate to avoid the pressure in the pipelines falling to vapour pressure. The air volume should be sufficient to produce cushioning effect to positive surge pressures. In essence, air chambers can usually be designed to be more compact than surge tanks. Air chamber has the demerits that regular maintenance has to be carried out to check the volume of air and proper design of pressure level of gas has to be conducted.

## Level One (Core FAQs)

### ***Part II: Thrust Blocks***

**1. Why are hydrodynamic forces not considered in the design of thrust blocks? (DTB1)**

Liquids in motion produce forces whenever the velocity or flow direction changes. The forces produced by changes in direction of fluid is called hydrodynamic forces and is equal to (density of fluid x discharge x change in flow velocity).

In underground pressurized pipelines, the configuration of pipelines causes unbalanced forces of hydrostatic and hydrodynamic and joint separation shall result if these forces are not properly balanced. In general, the unbalanced hydrostatic and hydrodynamic forces are called thrust forces. In normal applications of pressurized pipelines in wastewater works and waterworks, it is observed that the range of fluid velocity and discharge is quite limited. As such, the resulting unbalanced hydrodynamic forces induced are insignificant when compared with unbalanced hydrostatic forces and they are often neglected in the design of thrust blocks.

**2. How can thrust blocks resist unbalanced force in horizontal bends in watermain? (DTB2)**

Thrust block resists the unbalanced force in two common approaches. In the first approach, thrust block serves as gravity block which makes use of its own dead weight to resist the thrust forces. An example of this application is vertical down bends.’

The second approach of thrust block to resist unbalanced forces in watermain involves providing a larger bearing area so that the resulting pressure against the soils does not exceed the bearing capacity of soils. Therefore, the function of thrust block in this case is to make use of stiffness of concrete to spread the thrust force into larger area. An example of this application is horizontal bends in watermain.

**3. How can restrained joints resist thrust forces in pressurized pipelines? (DTB3)**

The unbalanced thrust forces in pressurized pipelines cause the line to move and joints to separate unless the unbalanced force is counterbalanced by some means such as thrust blocks.

Restrained joints can be adopted to resist the thrust forces. The mechanism of restrain joint involves gripping and locking the pipe joints together to avoid axial movement and joint separation. For the unbalanced thrust forces, they are distributed to the surrounding soils in such as way that the bearing area is assumed to decrease linearly from the location of thrust forces to the end of restrained pipes. The soil bearing against the pipelines and soil friction provide resistance to movement of pipelines.

**4. Is reinforcement necessary in thrust blocks? (DTB4)**

In normal situation, reinforcement is not required for thrust blocks in pressurized pipelines. However, certain amount of reinforcement has to be added in thrust blocks in the following situations [65]:

(i) The structure integrity of huge thrust block could be enhanced by the introduction of reinforcement.
(ii) At the anchorages for straps in thrust blocks, some reinforcement has to be designed to avoid the development of tensile stresses.

**5. Should pipelines be completely embedded into thrust blocks?**

For unreinforced concrete thrust blocks in bends and tees for pressurized pipelines, it is recommended that the contact surface between the pipelines and concrete thrust blocks should not exceed $45^o$ from either side of the pipe in the direction of thrust force through the center of pipelines [65]. The reason is to prevent the occurrence of potential cracking arising from the deformation of pipelines under loading condition. If it is necessary to embed the whole section of pipelines into concrete, it is suggested to coat the pipe with a flexible material.

# Level Two (Advanced FAQs)

## *Part I: Water Retaining Structures and Reservoirs*

### 1. What is the purpose of providing service reservoirs?

Service reservoirs, other than normal reservoirs, are provided because of the following reasons:

(i) In case of the breakdown of pumping stations and water treatment plants, it provides a temporary storage of water in emergency situation like fire fighting.
(ii) Since the demand of water supply from customers varies with time, the provision of service reservoirs aims to balance the fluctuation rate of water demand.
(iii) It provides a constant head of water to the distribution system under the design pressure.
(iv) In the event of the occurrence of water hammer or surge during the rapid closure and opening of pumping stations, the reservoir acts to attenuate the surge and performs like a surge tank.
(v) It leads to a reduction of the size of pumps and trunk mains connecting to the distribution system as the pumps are not required to directly cope with the peak rates of water demand by the introduction of service reservoirs. As such, there is substantial cost savings arising from the use of smaller pumping pipelines and smaller pumps.

### 2. Why do BS8007 specify the allowable crack width of water retaining structure as 0.2mm for severe or very severe exposure? (DWRS1)

For crack width less than 0.2mm, it is assumed that the mechanism of autogenous healing will take place in which the crack will automatically seal up and this would not cause the problem of leakage and reinforcement corrosion in water retaining structure.

When the cracks are in inactive state where no movement takes places, autogenous healing occurs in the presence of water. However, when there is a continuous flow of water through these cracks, autogenous healing would not take place because the flow removes the lime. One of the mechanisms of autogenous healing is that calcium hydroxide (generated from the hydration of tricalcium silicate and dicalcium silicate) in concrete cement reacts with carbon dioxide in the atmosphere, resulting in the formation of calcium carbonate crystals. Gradually these crystals accumulate and grow in these tiny cracks and form bonding so that the cracks are sealed. Since the first documented discovery of autogenous healing by the French Academy of Science in 1836, there have been numerous previous proofs that cracks are sealed up naturally by autogenous healing. Because of its self-sealing property, designers normally limit crack width to 0.2mm for water retaining structures.

**3. What is the purpose of setting indirect tensile strength in water-retaining structures? (DWRS2)**

The crack width formation is dependent on the early tensile strength of concrete. The principle of critical steel ratio also applies in this situation. The amount of reinforcement required to control early thermal and shrinkage movement is determined by the capability of reinforcement to induce cracks on concrete structures. If an upper limit is set on the early tensile strength of immature concrete, then a range of tiny cracks would be formed by failing in concrete tension [4]. However, if the strength of reinforcement is lower than immature concrete, then the subsequent yielding of reinforcement will produce isolated and wide cracks which are undesirable for water-retaining structures. Therefore, in order to control the formation of such wide crack widths, the concrete mix is specified to have a tensile strength (normally measured by Brazilian test) at 7 days not exceeding a certain value (e.g. $2.8N/mm^2$ for potable water).

**4. What is the importance of critical steel ratio in calculating thermal reinforcement? (DWRS3)**

The fulfillment of critical steel ratio means that in construction joints or planes of weakness of concrete structure, steel reinforcement will not yield and concrete fails in tension first. This is important in ensuring formation of more cracks by failure of concrete in tension, otherwise failure in steel reinforcement would produce a few wide cracks which is undesirable.

**5. In the design of service reservoirs, horizontal reinforcement in walls of reservoirs is placed at the outer layer. Why?**

Since service reservoirs are designed as water-retaining structures with stringent requirement of crack width control, the design of reinforcement of service reservoirs is under the control of serviceability limit state. For the walls of service reservoirs, contraction and expansion of concrete are more significant in the horizontal direction of walls because of their relatively long lengths when compared with heights. In this connection, in order to minimize the usage of reinforcement, horizontal bars are placed at the outmost layer so that the distance of reinforcement bars to concrete surface is reduced. Since the shorter is the distance to the point of concern, the smaller is the crack width and hence with such reinforcement arrangement advantages are taken if the reinforcement bars in the critical direction are placed closest to concrete surface.

**6. When designing a water storage tank, should movement joints be installed? (DWRS4)**

In designing water storage tanks, 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.

Let's take an example to illustrate this. For 30m long tanks wall, for a seasonal variation of 35 degree plus the hydration temperature of 30$^{o}$C, the amount of cracking is about 8.8mm. It can either be reduced to 0.3mm with close spacing or can be absorbed by movement joints. Anyway, the thermal movement associated with the seasonal variation of 35$^{o}$C is commonly accounted for by movement joints.

For water-retaining structure like pumping stations, the crack width requirement is even more stringent in which 0.2mm for severe and very severe exposure is specified in BS8007. It turns out to a difficult problem to designers who may choose to design a heavy reinforced structure. Obviously, a better choice other than provision of bulky reinforcement is to allow contraction movement by using the method of movement joints together with sufficient amount of reinforcement. For instance, service reservoirs in Water Supplies Department comprise grids of movement joints like expansion joints and contraction joints.

**7. Should mild steel or high yield steel be adopted as reinforcement of water-retaining structures? (DWRS5)**

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.

**8. In the design of service reservoirs, how are reservoir floors designed to prevent leakage of water due to seasonal and shrinkage movements?**

There are in general two main approaches in designing floors of service reservoirs:

(i) In the first method, movement joints are designed in each panel of reservoir floors so that they can expand and contract freely. Each panel is completely isolated from one another and a sliding layer is placed beneath them to aid in sliding.

(ii) The second method, on the contrary, does not make provision to free movement. Due to seasonal and shrinkage movements, cracks are designed to occur in the reservoir floors such that very tiny cracks are spread over the floor and these cracks are too small to initiate corrosion or leakage. However, in this case, the amount of reinforcement used is much larger than the first approach.

**9. In selection of waterstop, shall engineers use plain dumb-bell type or center-bulb type? (DWRS6)**

The plain dumb-bell type 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. Reference is made to W. L. Monks (1972).

**10. What is the purpose of uniform rate of application of water (i.e. 2m depth in 24 hours) in watertightness test of water retaining structures? (DWRS7)**

In watertightness test of water retaining structures, it normally requires the filling of water at a uniform rate or letting the water pool stand alone for some time before actual measurement is carried out. The reason for such provision in watertightness test is to allow sufficient time for water absorption to take place in concrete. After allowing sufficient time for water absorption to occur, the subsequent measurement of a fall in water level is deemed to be caused by water leakage instead of water absorption provided that adjustment has been made to other external effects such as evaporation.

**11. What is the purpose of adding cooling pipes or even using cold water for concrete in concreting operation?**

All these measures aim at reducing the placing temperature and reducing thermal cracks induced during concreting of massive pours. Since the final concreting temperature should be the ambient temperature, reducing the initial placing temperature will also lower the peak hydration temperature. Therefore, the temperature difference between the hydration peak and the ambient temperature is reduced accordingly and subsequently the thermal effect to concrete structure can be reduced by controlling the placing temperature.

# Level Two (Advanced FAQs)

## ***Part II: Pumps and Pumping Station***

### 1. What is the difference in arranging pumps in series and in parallel?(p1)

For identical pumps with similar functions, if the pumps arranged in series, the total head is increased without a change to maximum discharge. On the other hand, for pumps arranged in parallel to one another, the discharge is increased without any changes to maximum head.

### 2. Why are radial flow pumps suitable for small flows and high heads? (P2)

In radial flow pumps, a diffuser/volute is normally designed at it outlet to convert the kinetic energy gained during the pumping process to pressure head. The diffuser is characterized by widening of outlet pipes, resulting in the decrease of velocity (by continuity equation) and an increase in pressure head (by Bernoulli’s equation). In case of large flows to be handled by the pumps, the large velocity results in formation of significant Coriolis force which tends to deviate the outlet flow from design conditions.

At the inlet part of the pumps, the inlet size is smaller than the diameter of the impeller. Consequently, the velocity of flow associated with a small area is relatively large and there is less problem of separation in low flow condition. All in all, the efficiency of radial flow pumps is high when handling small flows.

### 3. Why are axial flow pumps suitable for large flows and low heads? (P3)

It is well known that axial flow pumps are most suitable for providing large flows and low heads. The reason behind this is closely related to the configuration and design of the pumps. In axial flow pumps, the size of inlet diameter is greater than that of impeller diameter. For low flow condition the velocity is relatively small and this increases the chances of occurrence of separation which brings about additional head losses and vibration. On the contrary, if the discharge is large enough the problem of separation is minimized.

**4. In terms of pumping performance, how should engineers determine the use of radial flow pumps and axial flow pumps? (P2 & P3)**

Specific speed is usually defined for a pump operating at its maximum efficiency. In order to minimize the cost of future operation, it is desirable to operate the pumps as close to the maximum efficiency point as possible. The specific speed for radial flow pumps is relatively small when compared with that of axial flow pumps. This implies that radial flow pumps tend to give higher head with lower discharge while axial flow pumps tend to give higher discharge with lower head.

**5. What is the difference in function between backward curved vanes and forward curved vanes in pumps? (P4)**

The power of a pump is related to discharge as follows:

$$Power = k_1 Q + \frac{k_2 Q^2}{\tan A}$$

where $k_1$ and $k_2$ are constants, Q is discharge and A is the angle between the tangent of impeller at vane location and the tangent to vane.

For A less than $90^o$ (forward curved vanes) it is unstable owing to unrestricted power growth. Large losses result from high outflow velocity. The preferred configuration is achieved when A is more than $90^o$ (i.e. backward curved vanes) because it has controlled power consumption and presents good fluid dynamic shape.

**6. What is the difference between best efficiency point and operating point for pumps? (P5)**

In a pumping system, a system curve can be derived based on the static head required to lift up the fluid and variable head due to possible head losses. The pump curves which relate the performance of the pumping to head against discharge can be obtained from pump suppliers. When the system curve is superimposed on the pump curve, the intersection point is defined as the operating point (or duty point). The operating point may not be necessarily the same as the best efficiency point. The best efficiency point is a function of the pump itself and it is the point of lowest internal friction inside the pump during pumping. These losses are induced by adverse pressure, shock losses and friction.

Losses due to adverse pressure gradient occur in pumps as the pressure of flow increases from the inlet to the outlet of pumps and the flow travels from a region of low pressure to high pressure. As such, it causes the formation of shear layers and flow separation. Flow oscillation may also occur which accounts for the noise and vibration of pumps. The effect of adverse pressure gradient is more significant in low flow condition.

For shock losses, they are induced when the inflow into pumps is not radial and contains swirl. In an ideal situation, the flow within the pump should be parallel to the impellers such that the flow angle is very close to the impeller angle. The deviation of the above situation from design causes energy losses and vibration.

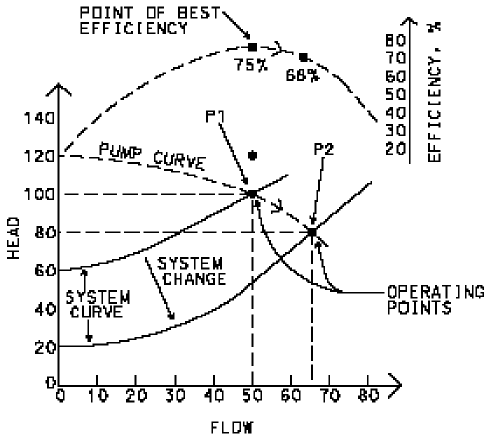



**Fig. A diagram showing point of best efficiency≠operating point**

## 7. What is the minimum volume of sump volume for pumps?

$$Maximum\ pumping\ rate = Q_p$$

$$Volume\ of\ sump = V$$

$$Inflow\ Rate = Q_i$$

$$Cycle\ Time\ T_c = t_1 + t_2$$

$$t_1 = \frac{V}{Q_p - Q_i}$$

$$t_2 = \frac{V}{Q_i}$$

$T_c = Minimum\ cycle\ time$

$$V = T_c / \left( \frac{1}{Q_p - Q_i} + \frac{1}{Q_i} \right)$$

$For\ Minimum\ Volume,$

$$\frac{\partial V}{\partial Q_i} = \frac{\partial}{\partial Q_i} \left\{ T_c / \left( \frac{1}{Q_p - Q_i} + \frac{1}{Q_i} \right) \right\}$$

$Minimum\ volume\ occurs\ when\ Q_i = 0.5 Q_p$

$$Hence,\ Minimum\ volume\ of\ sump = T_c / \left( \frac{1}{Q_p - 0.5Q_p} + \frac{1}{0.5Q_p} \right) = \frac{T_c Q_p}{4}$$

**8. In selecting screw pumps in polder scheme projects, what are the factors that affect the design capacity of screw pumps? (P7)**

The commonly used angles of inclination for screw pumps are $30^o$, $35^o$ and $38^o$. For screw pumps of relatively high lifting head, like over 6.5m, angle of inclination of $38^o$ is normally used. However, for relatively lower head and high discharge requirement, angle of inclination of $30^o$ shall be selected. In general, for a given capacity and lifting head, the screw pump diameter is smaller and its length is longer for a screw pump of $30^o$ inclination when compared with a screw pump of $38^o$ inclination.

To increase the discharge capacity of screw pumps, a larger number of flights should be selected. In fact, screw pumps with 2 flights are more economical that that with 3 flights in terms of efficiency and manufacturing cost. Moreover, the discharge capacity is also determined by the screw pump diameter and sizes of 300mm to 5000mm are available in current market.

**9. What are the components of a waterproofing system in the roof of a typical pumping station? (PS1)**

In the design of a waterproofing system at the roof of a pumping station, normally the following components are:

(i) Above the structural finish level of the concrete roof, a screed of uniform thickness is applied to provide a smooth surface for the application of waterproofing membrane. (Screed of varying thickness can also be designed on the roof to create a slope for drainage.) The screed used for providing a surface for membrane should be thin and possess good adhesion to the substrate. Moreover, the screed aids in the thermal insulation of the roof.
(ii) Above the screed, waterproofing membrane is provided to ensure watertightness of the roof.
(iii) An insulation board may be placed on top of waterproof membrane for thermal insulation. In cold weather condition where the loss of heat at the roof is significant, the insulation board helps to reduce these losses. On the contrary, in summer the roof is heated up by direct sunlight and the insulation layer reduces the temperature rise inside the pumping station.

**10. In pumping stations one of the choices for the material of water tanks is fibre-reinforced plastic (FRP). What are the advantages associated with this kind of material?**

There are two main advantages for FRP water tanks:

(i) It possesses high strength to weight ratio and this leads to the ease of site handling.
(ii) It is highly resistant to corrosion and thus it is more durable than steel water tank.

**11. What is the difference among cement plaster, cement render and cement screed? Under what situations should each of the above be used? (PS2)**

The purpose of plastering, rendering and screeding is to create a smooth, flat surface to receive finishes like paint, wallpaper etc.

Plastering is the intermediately coating of building materials to be applied on the internal facade of concrete walls or blockwalls.
Rendering is the intermediate coating for external walls only.
Screeding is the coating laid on floors to receive finishes like tiles, carpet, and marble...

Hence, these terms differ basically from the locations at which they are applied. Due to different locations of application of plasterwork, the proportion of material component for plaster and render is different. For example:

(i) Cement plaster
Undercoat- cement:lime:sand (by volume) = 1:4:16
Finishing coat - cement:lime:sand = 1:12:30

(ii) Cement render
Undercoat- cement:lime:sand (by volume) = 1:2:6
Finishing coat - cement:lime:sand = 1:3:6

**12. In the design of corbel beams in a pumping station, why are shear links designed in the top 2/3 of the section?**
**What is the general advice on the design? (PS3)**

Corbel beams are defined as z/d<0.6 where z is the distance of bearing load to the beams' fixed end (or called shear span) and d is depth of beams. The design philosophy is based on strut and tie system. To establish the design model, it is firstly assumed the failure surface, i.e. shear cracks extending to 2/3 of depth of beam. Experiment results verified that the failure cracks extended only to 2/3 of beam while the remaining 1/3 depth of concrete contributed as concrete strut to provide compressive strut force to the bearing loading.

Horizontal links are normally provided to corbel beams because experimental results indicated that horizontal links were more effective than vertical links when shear span/depth is less than 0.6. For shear span/depth>0.6, it should be not considered as corbel beams but as cantilevers.

In designing corbel beams, care should be taken to avoid bearing load to extend beyond the straight portion of tie bars, otherwise the corners of corbel beams are likely to shear off. Reference is made to L. A. Clark (1983).

**13. In the construction of pump troughs for accommodation of screw pumps, what is the construction method to ensure close contact between the screw pumps and the pump trough? (P7)**

In the construction of screw pump troughs, trapezoidal-shaped troughs are usually formed by using normal formwork. In order to enhance close contact between screw pumps and troughs, upon lifting the screw pumps into the troughs screeding works is carried out. Screw pumps are set to rotate and screeds are placed between the gap of screw pumps' blade and trapezoidal-shaped troughs during the rotating action of screw pumps. After the screed sets, it serves to prevent leakage of water during the pumping operation of screw pumps.

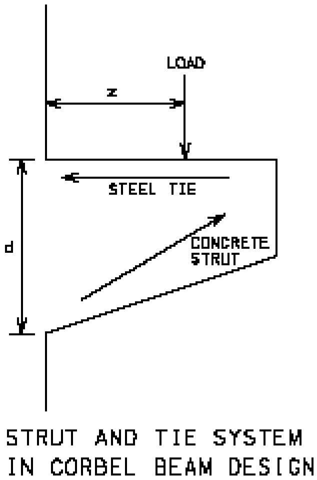



**Fig. Corbel Beam**

**14. Given a 1m high staircase resting on solid concrete, would it be adequate to design nominal reinforcement for the staircase?**

For the design of staircase, there are three main scenarios:

(i) Stairs spans longitudinally
This kind of stairs refers to stairs spanning between landings only without any side supports. In this case, the staircase should be designed as a beam between two end supports (i.e. landing) and the main reinforcement is provided at the bottom of staircase slabs.

(ii) Stairs spanning transversely
This kind of staircase is supported by sidewalls only and it may also be

supported by stringer beams. For the case of sidewalls, it acts as a cantilever beam and the main reinforcement are provided the top surface of slab. For the case of staircase supported sideways by both sidewall and stringer beam, it should be designed transversely with end supports as sidewall and stringer beam and reinforcement is provided at the bottom of the staircase.

(iii) Stairs resting on solid support
For stairs resting on solid supports, only nominal steel reinforcement is provided to control thermal and shrinkage cracking.

**15. Why are voids filled with lightweight infilling material in raft foundation of pumping stations? (PS4)**

To reduce the dead load and hence to reduce the settlement of pumping stations, the voids inside the raft foundations are filled with light material. If instead concrete is placed inside these voids, it poses severe thermal cracking problem and drastically increases loads to pumping stations. The use of general fill is also not desirable because its self-weight is comparable to that of concrete. On the contrary, if these voids are left vacant, water may penetrate into these voids during future operation and increases the dead load of pumping stations during its normal operation. Therefore, lightweight infill material, which is non-water-absorbing and non-biodegradable, is designed inside these voids to avoid ingress of water and to reduce the dead load of the structure.

# 12. Module Eleven: Steelworks

## Objectives

| Element | Description | Objective No. |
|---|---|---|
| **Bolts and Fasteners** | | |
| Bolts and Fasteners | Difference between fasteners, bolts and screws | BF1 |
| | Difference between normal bolts and high friction grip bolts | BF2 |
| | High strength friction grip (HSFG) bolts | BF3 |
| | Washers | BF4 |
| **Welding** | | |
| Design of Welding | Rules | DW1 |
| | Convex fillet welds | DW2 |
| | Square-groove, V-groove, U-groove and J-groove | DW3 |
| | Adjoining root faces in butt weld | DW4 |
| | Residual stresses | DW5 |
| | Ultrasonic test, radiographic inspection and magnetic particle flaw detection test | DW6 |
| | Acetylene gas cylinder | DW7 |
| **Miscellaneous** | | |
| Steel Reinforcement | Grouping reinforcement | SR1 |
| | Bend test | SR2 |
| | Re-bend test | SR3 |
| | Fatigue | SR4 |
| Steelworks | Square hollow section/circular hollow section | S1 |
| | Vibration in driving sheetpiles | S2 |
| | Wide and narrow sheetpiles | S3 |
| | Z-type, U-type, flat web and Pan-type of sheetpiles | S4 |
| | Castellated beams | S5 |
| | Corrosion inhibitors | S6 |
| | Fire protection | S7 |
| | Pedestals | S8 |
| | Stainless steel | S9 |

| Element | Description | Objective No. |
|---|---|---|
| | Hot dip galvanizing | S10 |

# Level One (Core FAQs)

## *Part I: Bolts and Fasteners*

### 1. What is the difference between fasteners, bolts and screws? (BF1)

Fastener is a general term to describe something which is used as a restraint for holding things together or attaching to other things.

The main physical distinction between screws and bolts is that screws are entirely full of threads while bolts contain shanks without threads. However, a better interpretation of the differences between the two is that bolts are always fitted with nuts. On the contrary, screws are normally used with tapped holes.

### 2. What is the difference between normal bolts and high friction grip bolts? (BF2)

High friction grip bolts are commonly used in structural steelwork. They normally consist of high tensile strength bolts and nuts with washers. The bolts are tightened to a shank tension so that the transverse load across the joint is resisted by the friction between the plated rather than the bolt shank's shear strength.

### 3. What are the advantages of using high strength friction grip (HSFG) bolts when compared with normal bolts? (BF3)

HSFG bolts have the following advantages when compared with normal bolts [47]:

(i) The performance of preloaded HSFG bolts under fatigue loading is good because the prestressed bolts are subjected to reduced stress range during each loading cycle when compared with unloaded bolts.
(ii) For structures adjacent to machinery which generate substantial vibration, preloading bolts can help to avoid the loosening of bolts.
(iii) HSFG bolts are used in connections where any slight slip movement would render the integrity of the whole structures break down.
(iv) Owing to its high tensile strength, it is commonly used in connections which require the taking up of high flexure and the tensile stress generated could be readily resisted by it high tensile strength.

## 4. Are washers necessary for proper operation of bolts? (BF4)

"Fastener" is a general term used to describe something which is used as a restraint for holding things together or attaching them to other things.

The main physical distinction between screws and bolts is that screws are entirely full of threads while bolts contain shanks without threads. However, a better interpretation of the differences between the two is that bolts are always fitted with nuts. On the contrary, screws are normally used with tapped holes.

High friction grip bolts are commonly used in structural steelwork. They normally consist of high tensile strength bolts and nuts with washers. The bolts are tightened to a shank tension so that the transverse load across the joint is resisted by the friction between the plates rather than the bolt shank's shear strength.

The purpose of installing washers in a typical bolting system is to distribute the loads under bolt heads and nuts by providing a larger area under stress. Otherwise, the bearing stress of bolts may exceed the bearing strength of the connecting materials and this leads to the loss of preload of bolts and the creeping of materials. Alternatively, flanged fasteners instead of using washers could be adopted to achieve the same purpose.

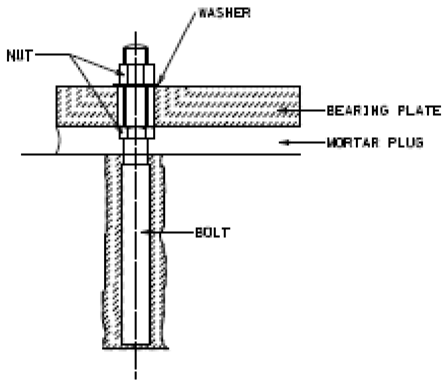



**Fig. Washer**

## 5. What is the behaviour of internal force in preloaded fasteners?

The force in a bolt in a bolted joint depends on the preloading force applied to it during the tightening operation. For instance, when the preloaded bolt is tightened with a certain force, the bolts' internal force will not increase significantly if the external applied force on the bolted joint does not exceed the preloading force. It looks like the bolt does not feel the external applied force and it is not until the external force has exceeded the preloading force when a substantial increase of internal force of the bolt will occur.

**6. Why are insulating washers installed between steel bolts and connecting aluminium plates? (BF4)**

Corrosion of aluminium can be triggered by putting it in contact with another metal in the presence of water. This is known as bimetallic corrosion or galvanic corrosion. The mechanism of such corrosion is the formation of a cell in moist condition so that an electric current is generated to flow between the two metals in direct contact. The degree of corrosion is influenced by the nature of connecting metals, their electrode potential, their areas, conductivity of fluid etc.

When aluminium plates are connected together by means of steel bolts, bimetallic corrosion may occur. Where there is presence of a good electrolyte like seawater, there may be local attack on aluminium. Therefore, some jointing compound or insulating insert and washer are adopted to insulate electrically the dissimilar metals from one another [1].

# Level One (Core FAQs)

## ***Part II: Welding***

### 1. There is a general rule in fillet weld that *"the leg should be equal to the thickness of metals."* Why? (DW1)

Let's take an example of 6mm thick plates to illustrate the rule. In case 12mm leg is adopted in the fillet weld, the weld volume would be 3-4 times more than required. It would result in waste of weld metal and welder's time. Worse still, over-welding may weaken the structure and result in distortion owing to the formation of residue stress. As such, the resulting weld could support less stress than fillet weld with "the leg equal to the thickness of metals."

On the other hand, for welding the same 6mm thick plates, if 3mm leg is used instead, it is under-welded. The resulting weld may break through the leg of the weld.

### 2. Convex fillet welds are sometimes used in welding. Why? (DW2)

There are three types of fillet weld cross section profile, namely, flat, convex and concave. The convexity in convex fillet welds serve as reinforcement, which is believed to provide additional strength. However, care should be taken in not introducing excessive convexity to fillet welds. Excess convexity leads to an increase in stress in weld toes which may subsequently fails by cracking.

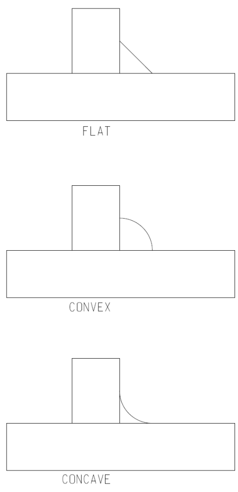



**Fig. Different types of fillet welds**

**3. In welding design what are the different applications of square-groove, V-groove, U-groove and J-groove? (DW3)**

When the base metal is thin (i.e. 0.125” to 0.25”), square-groove can be adopted. Where welding is carried out from one side of the joint, a temporary backup strip can be used to ensure proper joint penetration and to avoid excessive leakage of metal though the joint.

V-groove is commonly used for medium to thick metals (i.e. 0.25” to 0.375”). For even thicker metal plates, U-groove and J-groove can be adopted to provide good penetration of welded metal. One demerit of U-groove and J-groove is the preparation of the metal plates. For instance, air carbon arc and special mechanical cutting tools are required for preparing the joints.

**4. What is the function of gap between adjoining root faces in butt weld? (DW4)**

In the design of butt weld strength, it is generally assumed that its strength is at least equivalent to the parent metal. To enhance proper welding operation, the gap between two metals to be welded should not be too small, otherwise the root would be inadequately fused during welding and the butt weld strength would be reduced. On the other hand, the gap should be not set too large because the weld metal would simply pass through it. The function of the gap between adjoining root faces is to increase the depth of penetration down to the root of the weld.

However, it is not always possible to have access to both sides of the butt weld. Hence, the use of backing plates or rings can enhance the quality of welding from one side only. By inserting a backing plate inside the steel member, the correct alignment could be maintained and certain amount of tolerance on longitudinal fit can be permitted [47].

**5. Why are residual stresses incurred in steel after welding? (DW5)**

Considerable residual stresses are induced in connecting steel members after the welding operation. The local temperature of steel where welding takes place is higher than the remaining parts of the connecting steel members. This causes thermal expansion locally during welding and the subsequent contraction after welding. Tensile stresses associated with the thermal contraction generated during the cooling process are balanced by compressive stresses in the remaining parts of connecting steel members. As a result, residual stresses are induced during the welding operation.

**6. In checking the quality of weld, what are the pros and cons of various non-destructive weld inspection methods i.e. ultrasonic test, radiographic inspection and magnetic particle flaw detection test? (DW6)**

Currently, there are three common non-destructive testing of weld, namely radiographic inspection, ultrasonic testing and magnetic flaw detection test.

The method of radiographic approach was used commonly in the past until the arrival of ultrasonic inspection technique. The major difference between the two is that ultrasonic testing detects very narrow flaws which can hardly be detected by radiographic method. Moreover, it is very sensitive to gross discontinuities. Tiny defects, which characterize welding problems, are normally not revealed by radiographic inspection.

Moreover, ultrasonic inspection possesses the advantages that it can accurately and precisely locate a defect as well as figure out its depth, location and angle of inclination.

In the past, it was expensive to adopt ultrasonic means for inspection. Nowadays, the rates for both inspection methods are comparable. Most importantly, the x-ray and gamma ray used in radiographs are radioactive and pose potential safety hazard to testing technicians on site. Reference is made to Paul G. Jonas and Dennis L. Scharosch.

Magnetic flaw detection test can only be used for checking flaws in any metallic objects. This method is commonly used for inspecting surface cracks and slightly sub-surface cracks. However, surface and sub-surface cracks can be readily detected by radiographs and ultrasonic inspection.

**7. Why is acetylene gas cylinder for gas welding to be erected in upright position? (DW7)**

Acetylene gas is commonly used for gas welding because of its simplicity in production and transportation and its ability to achieve high temperature in combustion (e.g. around 5,000$^{o}$F). Acetylene is highly unstable and flammable and would explode in elevated pressure when reacting with oxygen in air. Storing acetylene gas in cylinders under pressure is very dangerous. Gas acetylene used for welding purposes is stored in cylinders of liquid acetone contained in porous material (like firebrick). This is for cooling purpose in the event of thermal decomposition and to ensure that there is no free space left for acetylene gas. It also prevents the formation of high-pressure air pockets inside the cylinder. Dissolved acetylene in acetone will no longer be in contact with oxygen and is not subject to decomposition. Acetone is used because it is capable of dissolving large amount of acetylene gas under pressure without changing the nature of the gas.

The cylinders for gas welding i.e. oxygen cylinders and acetylene cylinders, when not in use should be stored separately because any mixture of these gases resulting from accidental leakage can be highly explosive. When in use, acetylene cylinders should always be kept in upright position. Otherwise, acetone liquid will be drawn from the cylinders with the gas if they are kept horizontally, resulting in significant leakage of acetone liquid will result.

Note: Oxygen and acetylene gas cylinders are commonly used in construction sites for gas welding.

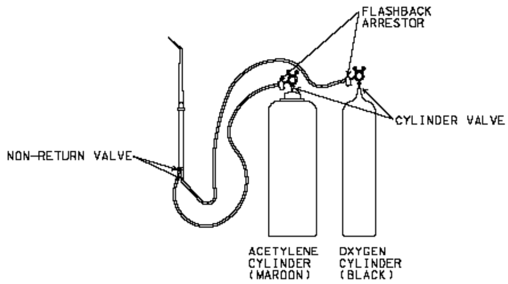



**Fig. Acetylene gas cylinder erected in upright position**

# Level Two (Advanced FAQs)

## *Part I: Miscellaneous*

### 1. What is the reason of grouping several reinforcement bars in concrete structure? (SR1)

For too much reinforcement to be incorporated in concrete structures, the reinforcement bars are sometimes groups because:

(i) It facilitates placing of fresh concrete with more space available;
(ii) It tends to limit segregation , which otherwise caused by close spacing of bars;
(iii) It ensures good cover.

### 2. What is the purpose of conducting bend test of reinforcement? (SR2)

It is not uncommon that steel reinforcement is bent prior to installation into concrete structure. However, upon bending process steel reinforcement may fracture owing to the following reasons:

(i) The ribs on steel bars serve as location of stress concentration which is a potential weak point for fracturing.
(ii) Owing to their intrinsic high strength, large force is required during the bending process.
(iii) The radius of bending is too tight.

Temperature is also an important factor for controlling the risk of steel fracture. The risk of fracture is increased when there is a drop of temperature because steel has lower toughness at low temperatures.

Therefore, bend test are carried out for reinforcing steel to testify their bending performance.

### 3. What is the purpose of conducting re-bend test of steel reinforcement? (SR3)

In BS4449:1997 it species a re-bend test of steel reinforcement where reinforcing steel is bent $45^o$ at $100^oC$ for an hour and then bent back by $23^o$. The purpose of re-bend test is to measure the effect of strain ageing on steel. Strain ageing has embrittlement effect which takes place after cold deformation by diffusion of nitrogen in steel. Hence, there is limitation stated in some design codes to restrict the nitrogen content of steel to 0.012%.

**4. Is fatigue more serious in large-diameter reinforcing steel? (SR4)**

Indeed past research showed that large-diameter reinforcing steel appeared to be weaker under fatigue loading conditions. Therefore, in some standards the stress range for testing fatigue of steel bars are reduced for increasing bar size for the same reason.

Moreover, fatigue performance of steel bars is also governed by stress concentrations at the root of ribs. Bar failure usually initiates from the root of ribs under fatigue loading. In this connection, any damage to ribs during bending process can lead to failure by fatigue.

**5. How can heating assist in rebending of steel reinforcement?**

It is not uncommon that starter bars are bent up within the formwork as a measure of temporary protection. Later, after the concrete is placed and formwork is removed, the steel bar reinforcement would be pulled out and straightened.

For rebending of bars, it is preferably be implemented for small diameter bars with mild steel. Moreover, rebending of steel bars should not be carried out below $5^{o}C$ owing to brittle fracture. Heating could be adopted to assist rebending process. However, heating should be applied to a good length of a bar instead of a concentrated location because of the possible occurrence of overheating. Moreover, after heating the cooler adjacent part of the steel bar may experience fracture when the bars are stressed in case concentrated heating is applied to steel bars.

**6. Why does square hollow section become more popular than circular hollow section in steelworks? (S1)**

Circular hollow section was available for many years until in 1960s for the approval of square hollows section and rectangular hollow section. From technical point of view, circular hollow section is the most efficient form of strut when compared with square hollows section and rectangular hollow section. However, nowadays, the use of square hollows section and rectangular hollow section is ever on the rise because of the ease of connections between individual struts.

**7. What is the vibration mechanism caused by driving sheetpiles? (S2)**

There are generally three main vibration mechanisms caused by driving sheetpiles:

(v) When the sheetpiles are impacted by a hammer, a compressive wave would be formed and it travels down to the toe of sheetpiles. A large amount of energy would be used to cause downward movement of sheetpiles while some of the energy would be reflected back up to the sheetpiles. The remaining energy would be transmitted to soils which expand outward as a spherical wavefront called "P" waves.
(vi) The impact action of hammer causes temporary lateral deformation of sheetpiles. A surface wave is then established which travels outward from pile shaft circumferentially.
(vii) The downward motion of sheetpiles arising from hammering action induces vertically polarized shear waves which propagate outward in cylindrical wavefront.

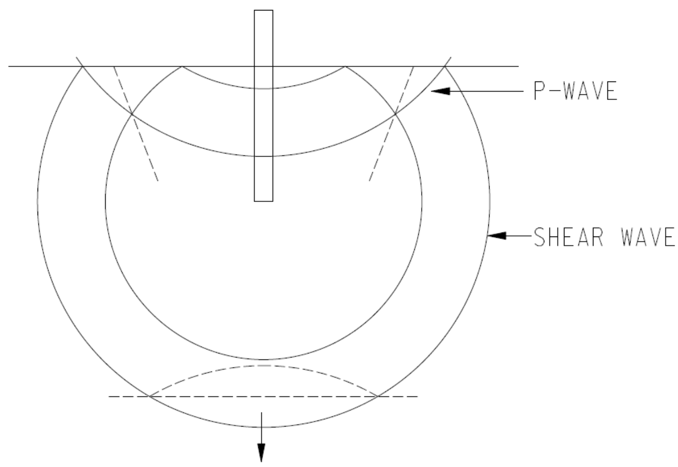



**Fig. Vibration waves by driving**

**8. Should wide or narrow sheetpiles be adopted in temporary work? (S3)**

In general, wide and deep sheetpiles tend to be more cost-effective than narrow sections because they provide the same bending strength with a lower weight per square foot. As such, with increasing width of sheetpiles sections, fewer sheetpiles are required to cover a certain length of piling operation. Hence, the cost of installation can be reduced accordingly.

However, consideration should also be given to the drivability of steel sections. The larger the surface area of piling sections, the higher the driving force is required. Therefore, the drivability of wide sheetpiles appears to be lower than that of narrow sheetpiles.

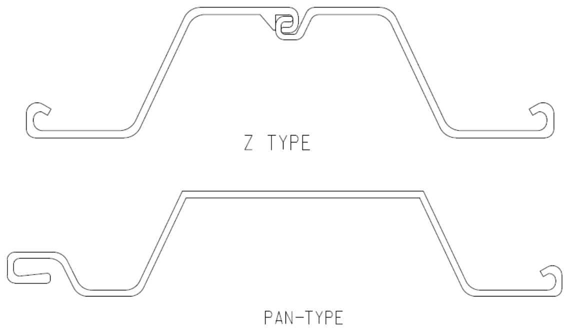
**9. What is the difference of the following types of sheetpiles (Z-type, U-type, flat web and Pan-type)? (S4)**

Z-type: The interlocks are situated as far away from the neutral axis as possible to facilitate good shear transfer and to enhance higher strength to weight ratio. This is the most common type of sheetpiles used in many countries.

U-type: U-type sheetpiles perform in similar manner as Z-type sheetpiles. The major difference between them lies on the location of interlocks. For U-type sheetpiles, the interlocks are located at neutral axis which reduces the efficiency of the section. The properties of U-type shall be decreased owing to the problem of shear transmission.

Flat web: The mechanism in resisting load differs from other types of sheetpiles. Flat web are usually installed in circles and the sheetpiles are held together by tensile strength of the interlock.

Pan-type: Pan-type sheetpiles are smaller in size than most other sheetpiles. Owing to their smaller size, they are commonly used for resisting short and light loaded structures.



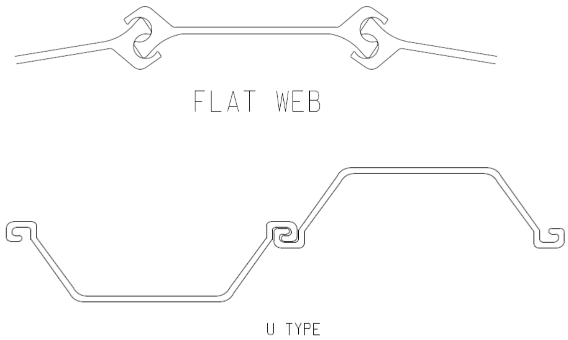



**Fig. Different types of sheetpiles**

## 10. What are the advantages in using castellated beams? (S5)

Castellated beams refer to the type of beams which involve expanding a standard rolled steel section in such a way that a predetermined pattern is cut on section webs and the rolled section is cut into two halves. The two halves are joined together by welding and the high points of the web pattern are connected together to form a castellated beam. The castellated beams were commonly used in Europe in 1950s due to the limited ranges of the available steel rolled section and the cheap labour cost. In terms of structural performance, the operation of splitting and expanding the rolled steel sections helps to increase the section modulus of the beams. Moreover, it is versatile for its high strength to weight ratio so that lighter section can be designed with subsequent cost saving in foundation [20].

## 11. How do corrosion inhibitors function? (S6)

Corrosion inhibitors are chemical substances that, when added in small concentrations, stop or reduce the corrosion or reaction of the metal with the environment. It normally functions by one or more of the following mechanisms [50]:

(i) It may alter the external environmental conditions by taking away or inactivating an aggressive agent;
(ii) It may adhere to form a film on the surface of the metal;
(iii) It brings about the formation of corrosion products.

## 12. Why should protection be implemented for steelwork against fire? (S7)

Owing to the high thermal conductivity of steel, the temperature of unprotected steel is almost the same as the temperature of fire. Since the yield strength of structural steel drops approximately by half when its temperature rises to about 550$^{o}$C, it is usually provided with some forms of insulation. The consequence of the outbreak of fire in proximity of unprotected structural steel is the potential loss of load carrying capacity of steel and the occurrence of substantial movement of steel.

**13. What are the functions of different components in a painting system?**

In a typical painting system, there are normally three main layers, primer, undercoat and finishing coat. The primer acts as the first coat of the painting system and adheres to the substrate. It serves to provide a foundation for other coats. The mid-coat, undercoat, is designed to increase the film thickness and hinder the background colour. Moreover, it aids in the reduction of permeability by incorporating pigments like micaceous iron oxide. Finally, the finishing coat contributes to the appearance of the painting system like colour. Sometimes, it may be designed to provide additional abrasive resistance. However, in terms of corrosion protection to steelworks, it does not add much value.

The main component which serves to inhibit corrosion is the primer because it is in direct contact with steel surface. In general the primer is pigmented with inhibitors like zinc and zinc phosphate which protect the steelwork by sacrificial protection [27]. Initially the primer is porous and the products generated by sacrificial protection of zinc fills these voids and the primer acts as a barrier.

**14. Why are holes sometimes present in the base plates connected to footings?**

The surface of footings is normally quite rough so that some leveling has to be carried out for the base plates. The interface between the base plates and footings after leveling is subsequently filled with grout. During grouting, trapping of air may occur at the underside of base plates and this leads to the formation of cavities and uneven contact surfaces on which the base plates are rested. As such, some holes may be drilled in the base plate to avoid the occurrence of air trapping [63].

**15. What is the purpose of pedestals? (S8)**

When structural steelworks are connected to the foundation, pedestals are normally designed to carry loads from metal columns through the ground surface to the footings which are located below the ground surface. With the installation of pedestals, it is the pedestals, instead of metals, which come into contact with soils. The purpose of the provision of pedestals is to avoid the direct contact of metal columns with soils which may cause possible metal corrosion by soils. The soils around the pedestals should be properly compacted to provide sufficient lateral resistance to prevent buckling of pedestals [9].

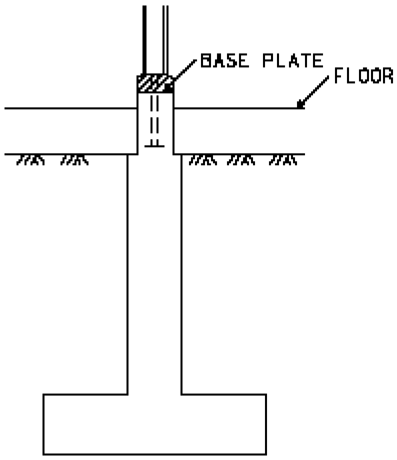



**Fig. Pedestal**

**16. Is stainless steel really stainless in construction application? (S9)**

Stainless steel refers to alloy steels with more than 10.5% of chromium and consists of several groups like austenitic, ferritic, martensitic etc. Austenitic stainless steel is normally used in structural applications because of its high corrosion resistance. Austenitic and ferritic types of stainless steel cover about 95% of stainless steel applications. Stainless steel is not stainless although it is corrosion resistant under a wide range of conditions.

A passive layer of chromium oxide is formed on stainless steel's surface which renders it corrosion resistant. This chromium oxide layer acts as a stiff physical barrier to guard against corrosion and makes it chemically stable. Moreover, when this layer is damaged, it can perform self repairing where there is a sufficient supply of oxygen. However, stainless steel will still corrode by pitting in marine environment where chloride attack occurs. Therefore, appropriate grades and types of stainless steel have to be selected in polluted and marine environment to minimize the problem of corrosion. Reference is made to Euro Inox and the Steel Construction

Institute (2002).

**17. What is the mechanism of protection by hot dip galvanizing? (S10)**

Hot dip galvanizing protects steel/iron from corrosion by:

(i) It forms a metallic zinc and zinc-iron alloy coating on top of steel surface. This zinc coating reacts with moisture in atmosphere to from zinc salts which act as an insulating layer for steel/iron.

(ii) Zinc is higher than steel/iron in the galvanic series and when these dissimilar metals with different electrical potential are in contact, the zinc anode corrodes and offers sacrificial protection to steel/iron and hence steel/iron is protected from corrosion.

# 13. Module Twelve: General Issue

## Objectives

| Element | Description | Objective No. |
|---|---|---|
| **General Issue** | | |
| General Issue | Working stress approach and limit state approach | GI1 |
| | Different components of paint | GI2 |
| | Insurance | GI3 |
| | Design life | GI4 |
| | Sureties and security | GI5 |
| | Safety | GI6 |

**1. What is the difference between working stress approach and limit state approach? (GI1)**

For working stress approach, service loads are used in the whole design and the strength of material is not utilized in the full extent. In this method of design, stresses acting on structural members are calculated based on elastic method and they are designed not to exceed certain allowable values. In fact, the whole structure during the lifespan may only experience loading stresses far below the ultimate state and that is the reason why this method is called working stress approach. Under such scenario, the most economical design can hardly be obtained by using working stress approach which is now commonly used in the design of temporary works.

For limit state approach, for each material and load, a partial safety factor is assigned individually depending on the material properties and load properties. Therefore, each element of load and material properties is accurately assessed resulting in a more refined and accurate analysis of the structure. In this connection, the material strength can be utilized to its maximum value during its lifespan and loads can be assessed with reasonable probability of occurrence. Limit state approach is commonly used for the majority of reinforced concrete design because it ensures the utilization of material strength with the lowest construction cost input.

**2. What are the functions of different components of paint? (GI2)**

For normal paint application, there are mainly three main components of paint, namely primer, undercoat and finishing coat.

Primer: This is the first layer of a typical painting system and it is used to inhabit corrosion and provide a good bond for subsequent coats.

Undercoat: This component acts as a barrier to corrosion agents and even out irregularities of bonding surface. It also serves to hide the underlying background and prevent the details and colour of the area of application to affect the designed colour and finishing details of paint.

Finishing coat: This is the final layer of a typical painting system and it protects the underlying layers from the effect of adverse weather conditions (e.g. sunlight) and to provide the designed properties of paint like colour, impermeability, wearing resistance, etc.

**3. If the contractor is liable for defective works for 12 years with contract under seal (6 years with contract not under seal), then what is the significance of Maintenance Period? (GI3)**

Defective works constitute a breach of contract in accordance with Limitation Ordinance (Cap. 347).

An action founded on simple contract (not under seal) shall not be brought after expiration of 6 years while an action founded with contract under seal shall not be brought after expiration of 12 years. For construction works, the date of counting these actions should be the date of substantial completion.

To answer the above question, one should note that under the contractual requirement, the contractor during Maintenance Period has the right to rectify the defects and the employer has also the right to request the contractor to make good defective work. However, after the expiry of Maintenance Period, in case of any arising of defects, the employer has to employ others to rectify these works and bring the action to court to claim the contractor for the costs associated.

**4. Is the procurement of third party insurance necessary to be incorporated in contract for construction works? (GI3)**

The purpose of third party insurance is to protect contractors from bankruptcy in case there are severe accidents happened to the third party due to the construction work. Therefore, in government contracts, contractors are requested contractually to procure third party insurance from the commencement of contract until the end of Maintenance Period. If contractors have the financial capability to handle the claims due to accidents to third party, the client is not bound to include this requirement in the contract.

**5. The insurance policy of insurance companies has changed recently. What is the major change? (GI3)**

Original Clause 4.6.1
“Liability in respect of death, bodily injury, illness or disease suffered by any person employed by an insured Contractor or employed by any party to whom part or parts of the insured Contract have been sub-contracted. However, this exclusion shall not apply to any liability which may attach to any sub-contractor insured under this Policy in respect of death, bodily

injury or illness or disease suffered by a person employed by any other sub-contractor."

Revised Clause 4.6.1
"Liability in respect of death, bodily injury, illness or disease suffered by:
1) Any person employed by any insured party i.e. principal contractors, sub-contractors, sub-sub-contractors for the purpose of execution of insured contract or any parts thereof and
2) Any person to whom part or parts of the insured contract have been sub-contacted including but not limited to self employed sub-contractors."

In essence, the original clause 4.6.1 has no cover for death/injury to employees of contractors or sub-contractors because they should have separate insurance cover under employee's compensation ordinance. However, it does not exclude the liability for "worker to worker" i.e. sub-sub-contractors. For the revised Clause 4.6.1, it rules out the liability for death/injury to employees of any insured party.

**6. Should design life be the same as return period for design conditions? (GI4)**

Design life means the minimum duration a structure is expected to last. The longer is the design life; the higher is the cost of a project. Therefore, in choosing the design life for a structure, engineers should consider the design life which generates a economical project without sacrificing the required function.

In selection of return period of certain design conditions, winds, waves, etc., one should consider the consequences of exceedance. In fact, there are normally no extreme maximum values of these design conditions and its selection is based on the probability of exceedance which is related to return period.

Therefore, design life may not be equal to return period of design conditions because their selections are based on different considerations.

**7. What is the difference between sureties and security? (GI5)**

In construction contracts, if a contractor fails to perform the works, the employer would suffer from severe financial loss and therefore some forms of protection has to be established in the contract.

For surety bond, the contractor obtains a guarantee from a third party i.e. a bank or an insurance company, which in return for a fee, agrees to undertake the financial responsibility for the performance of contractor's obligations. This third party will pay to the employer in case there is a contractor's default.

For security, a sum of money is deposited in the employer's account and upon satisfactory fulfillment of contractor's obligations, the sum will be released to the contractor.

**7. How does safety helmet function? (GI6)**

The main principle of safety helmet is to protect workers against impact by falling objects or struck by swinging object *by energy absorption*. Upon hitting by an object, the helmet dissipates some of the energy in the following mechanisms:

(i) Stretching of harness inside the helmet;
(ii) Partial damage of outer shell of helmet.

The remaining energy is then evenly spread around the head to reduce the hitting stress on worker's head.

**8. Working at height is commonly defined in many countries as falling more than 2m. Why? (GI6)**

For more than 2 metres, it is commonly defined as high level fall in which most injuries are resulted from. However, there is an increasing trend that there has been similar number of injuries from low level falls (i.e. less than 2 metres) and from high level falls. As such, some countries have deleted the *"2 metre rule"* as the definition of falling at height. Instead, it is newly defined as working at a place from which a person could be injured by falling from it, regardless of whether it is above, at or below ground level without stating the level of fall.

## ***About the Author***

**Vincent T. H. CHU (朱敦瀚) 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", "Civil Engineering Practical Notes A-Z" , "Ask Vincent Chu (Common FAQ on Practical Civil Engineering Works)", "The Underlying Reasons in Practical Civil Engineering Works" and "A Closer Look at Prevailing Civil Engineering Practice – What, Why and How".**