# An Explanatory Handbook to the Code of Practice for Structural Use of Concrete 2013

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## Foreword

To facilitate engineers to get thorough understanding and background information for design of reinforced concrete structures in Hong Kong, the first edition of handbook entitled "An explanatory handbook to the Code of Practice for Structural Use of Concrete 2004" was published in 2006 by the Structural Division of the Hong Kong Institution of Engineers. The code of practice was further revised after a five-year review work by the TC and the updated version was published. During the years, there were further amendments and currently it is in its 2020 edition. It is time to revise and update the handbook which is now named "An explanatory handbook to the Code of Practice for Structural Use of Concrete 2013". The revised handbook has conformed to the latest code of practice. It incorporated additional design examples and explanatory materials in order to provide practitioners guidance for practical design works.

It is my great honour to represent the Structural Division of the Hong Kong Institution of Engineers to express our sincere gratitude to the Working Committee, chaired by Ir Dr Simon Wong, and all committee members of structural division for revising this Handbook on the Code of Practice for Structural Use of Concrete 2013.

Ir Ben Tse
Chairman
Structural Division,
HKIE March 2022

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# 1 GENERAL

## 1.1 Scope

This code covers mainly the analysis, design and construction of reinforced and prestressed concrete buildings, where the concrete is made of normal-weight aggregate. Precast components are not covered; for the design of precast components, the Code of Practice for Precast Concrete Construction is referred to.

Any parts of the buildings subjected to highway loadings, which may be more appropriately regarded as highway structures or bridges, are also not covered in this code; for any such parts, a highway design code is referred to.

Although not explicitly stated, this code does not deal with earthquake loads or earthquake resistant design. Earthquake resistance is not normally a design requirement for Hong Kong. If earthquake resistance is demanded, it is recommended to consult other codes of practice for the design. Nevertheless, it is considered prudent to pay particular attention to the detailing of the concrete structures so as to provide a reasonable amount of ductility, which is needed not only for earthquake resistance, but also for robustness and overall safety of the structures.

## 1.2 References

This code makes reference to the other codes of practice currently in use in Hong Kong and the Hong Kong Construction Standards CS1, CS2 and CS3.

As for codes/standards elsewhere, it is not possible for these codes/standards to cover every aspect of concrete building structures. Where provisions cannot be found in these codes/standards, the engineer may resort to other acceptable codes/standards elsewhere. However, it should be borne in mind that the characteristics of concrete made in different places are not quite the same because of the differences in raw materials, curing conditions and concreting practices, and that the design standards in different places may vary quite substantially. Therefore, when applying codes/standards elsewhere to Hong Kong, the engineer will have to provide justification or evidence that the material parameters adopted are appropriate under the Hong Kong conditions and that the overall standard of the design is not inferior to that stipulated in this code.

## 1.3 Assumptions

Concrete is a variable material, of which the quality is sensitive not only to variations in the mix proportions, but also to the workmanship and the curing conditions. Hence, the reliability of the material is dependent on the quality control of the concrete production and the site supervision during concreting. Adequate quality control and site supervision have been assumed in this code.

The engineer responsible for site supervision should satisfy himself/herself that the concrete in the finished structure is of the required quality and keep all records of the concrete tests. If there is any special construction sequence or curing regime designed to mitigate cracking or to achieve certain special properties, it should be advisable to brief all the site staff involved beforehand and carry out monitoring to ensure strict adherence to the specified procedures.

The structural behaviour of a concrete structure can be fairly complicated. Even when the applied load is relatively small, the concrete structure might have already cracked. Hence, when estimating the stiffness of the concrete structure, the possibility of premature cracking should be considered. Moreover, a concrete structure has in general only finite and sometimes rather limited ductility, which is dependent on the concrete grade and the details of the reinforcement provided. Hence, when applying inelastic or plastic analysis, due considerations should be given to whether the concrete structure has sufficient ductility to meet the ductility demand. In any case, the engineer should ascertain himself/herself that the assumptions made in the structural analysis are valid. When unconventional materials or structural forms are used, it may be necessary to verify the validity of the design assumptions by rigorous analysis or even prototype testing.

It has been assumed in the code that the design is carried out by an appropriately qualified design engineer and that the structural modelling and design calculations contain no mistakes. It is the design engineer’s responsibility to arrange design scrutiny of the structural modelling and independent checking of the correctness and accuracy of all the design calculations. The various safety factors provided in the code are not supposed to cover any mistakes or errors in the structural modelling and design calculations.

## 1.4 Definitions

The various terms in the code may have specific meanings as defined in Section 1.4 of the code. Common sense interpretation should not be relied on and this section should always be referred to whenever there is doubt to the exact meaning of any term.

## 1.5 Symbols

The majority of symbols used are taken from BS8110: Parts 1 and 2. Other symbols are taken from Eurocode 2. Where the same symbol has been used in these two codes for more than one definitions, a new symbol has been defined so that each symbol used in the code has only one specific meaning.

# 2 BASIS OF DESIGN

## 2.1 Requirements

Performance standards

In simple technical terms, the basic performance requirements of a building structure are:

- *structural adequacy* (have sufficient strength and stability to resist the largest possible loads and sufficient endurance to resist repeated loads during the expected life);
- *serviceability* (free of excessive deformation, vibration and cracking etc);
- *durability* (able to last the expected life with only normal maintenance required);
- *fire resistance* (able to withstand fire up to a certain period of time);
- *robustness* (will not cause disproportionate collapse when subjected to local damage due to accidental overload or impact); and
- *ductility* (able to undergo significant plastic deformation before rupture).

The main function of the code is to set the design standards for the above performance requirements. However, it should be borne in mind that setting the design standards is actually a social-economical issue. None of the above performance requirements can be demanded or achieved in absolute sense. For instance, there is no absolute structural safety (encompassing both structural adequacy and robustness) in this world. No matter how high the safety standard is, there is always a probability, albeit small, that the structure will fail. The safety standard could be set higher so that the risk of failure would become smaller, but at the same time the cost of construction would soar and the structure might become excessively bulky to defeat its intended function. Likewise, there is no absolute durability because it is physically impossible for a structure to last forever. Therefore, in reality, it is necessary to draw a line somewhere and set performance standards that are high enough to ensure an appropriate degree of reliability and yet practicable and economical.

The design standards set in the code have been developed over a long period of time with the best of our engineering knowledge and skill embodied. They are generally comparable to most national standards, especially the British Standards, and may therefore be considered in line with international practices. On the whole, they represent a reasonable balance between the performance standards of buildings and the economic/sustainable development of the society that is so far acceptable to the general public.

The design standards stipulated in the code are recommended minimum standards for general buildings under normal conditions. If, for some reasons, such as high susceptibility to unusual loading conditions or simply high value/importance of the building, it is considered prudent to design the building structure to a higher standard, it is up to the design engineer to do so, with perhaps consent from the client.

Design working life

Regarding the expected life of the building, the code specifies the expected life in terms of the *design working life,* which is assumed to be 50 years for general buildings and other common structures. The usage of this terminology is explained below.

A number of different terminologies and definitions have been used for specifying the "life" of a building, namely, design life, service life and design working life. The various terminologies could be a bit confusing. The terms design life and service life are used in BS7543 while design working life is used in EN documents, as depicted below.

In BS7543: 1992,

- *design life* is the period of use intended by the designer (e.g. as stated by the designer to the client to support specification decisions),
- *service life* is the actual period of time during which no excessive expenditure is required on operation, maintenance or repair of a component, or re-construction, and
- *required service life* is the service life specified to meet users' requirements (e.g. as stated in the client's brief for a project or in the performance specification).

In EN documents, *design working life* is employed. This is the period of time during which a structure that has undergone normal maintenance is unlikely to require major repairs.

In the new code, the terms design life, service life and required service life defined in BS7543 are not used because in practice the design life should not be different from the required service life and the purposeful differentiation between design life and required service life would only cause confusion. Relatively, it should be simpler and better to just use the term design working life, which is more clearly defined, as in the EN standards.

Apart from the difference in terminologies, the British Standards and the EN documents also differ in the specified design life or design working life periods, as depicted below.

Among the various British Standards, BS8110 has never specified any design life. BS7543: 1992 recommends that (see Table 1 of the document) for category 4 normal life structures, such as new health and educational buildings, new housing and high quality refurbishment of public buildings, the design life is a minimum period of 60 years while for category 5 long life structures, such as civic and other high quality buildings, the design life is a minimum period of 120 years. In BS5400: Part 1: 1988, the design life of steel, concrete and composite bridges is recommended to be 120 years.
On the other hand, EN1990: 2002 (E) recommends that (see Table 2.1 of the document) for design working life category 4 structures, such as building structures and other common structures, the indicative design working life is 50 years while

for design working life category 5 structures, such as monumental building structures, bridges and other civil engineering structures, the indicative design working life is 100 years.

From the above, it can be seen that for normal buildings, the design life specified in BS7543 is 60 years whereas the design working life specified in EN1990 is 50 years. However, the difference between 60 and 50 years does not really carry much significance because up to now there is no rigorous scientific basis for these values. In actual practice, the environmental and usage conditions of the buildings could vary from one extreme to another and similar buildings designed to the same durability standard might end up with very different degrees of deterioration after 30 or 40 years. Hence, the specified design life or design working life should be treated only as a nominal value. A value of 50 years has been adopted in the new code. This does not mean that the buildings designed as per the new code would not last longer than 50 years. Like many existing buildings that have survived longer than 50 years, it is quite possible for the buildings, after major repairs or retrofitting, to survive up to 60 years or so.

## 2.2 Principles of limit state design

### 2.2.1 General

The new code is based on the limit state design philosophy. According to the limit state design philosophy, a structure has to be designed to satisfy not just one performance requirement but a multitude of performance requirements, or in other words, to be designed to stay within *limit states*, which define the bounds of acceptable performance limits, under all possible usage conditions.

Limit states to be considered in the design are mainly the ultimate limit state and the serviceability limit state. The ultimate limit state is related to structural safety whereas the serviceability limit state is related to proper functioning of the structure. The ultimate limit state is generally considered more important. Hence, most engineers choose to first design for the ultimate limit state, thinking that the ultimate limit state is likely to be the most critical limit state, and then check that the remaining limit states will not be reached.

However, the various structural modelling assumptions made during ultimate limit state design, such as the neglect of the torsional stiffness of frame members and the neglect of the out-of-plane stiffness of shear walls, which are on the conservative side from structural safety point of view, are not necessarily on the conservative side when applied to serviceability limit state design. When the serviceability limit state is considered, the torsional stiffness of frame members and the out-of-plane stiffness of shear walls should not be neglected because the torsional moment and out-of-plane bending moment induced may cause cracking of the concrete. The common practice of using the structural model originally developed for ultimate limit state design also for serviceability limit state design may not be acceptable.

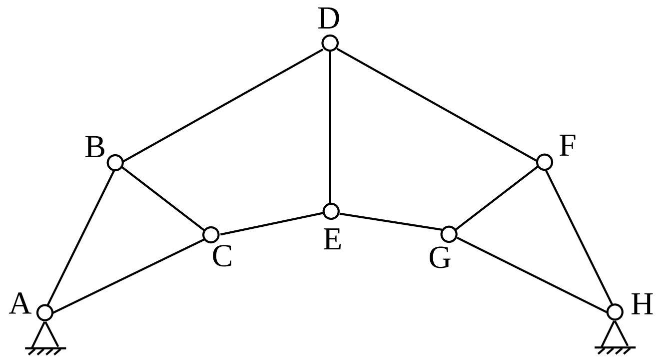
### 2.2.2 Ultimate limit state (ULS)

ULS design is concerned mainly with *strength*, *stability* and *robustness* of the structure, although collapse, overturning and buckling have also been mentioned as if they are something different in Clause 2.2.2.1 of the code (somehow robustness has been missed out in this clause). Collapse may be a consequence of strength, stability or robustness failure while overturning and buckling are both consequences of stability failure. Hence, collapse, overturning and buckling are only duplicated concepts. It should be more systematic to just concentrate on strength, stability and robustness, which already cover all aspects of ULS design.

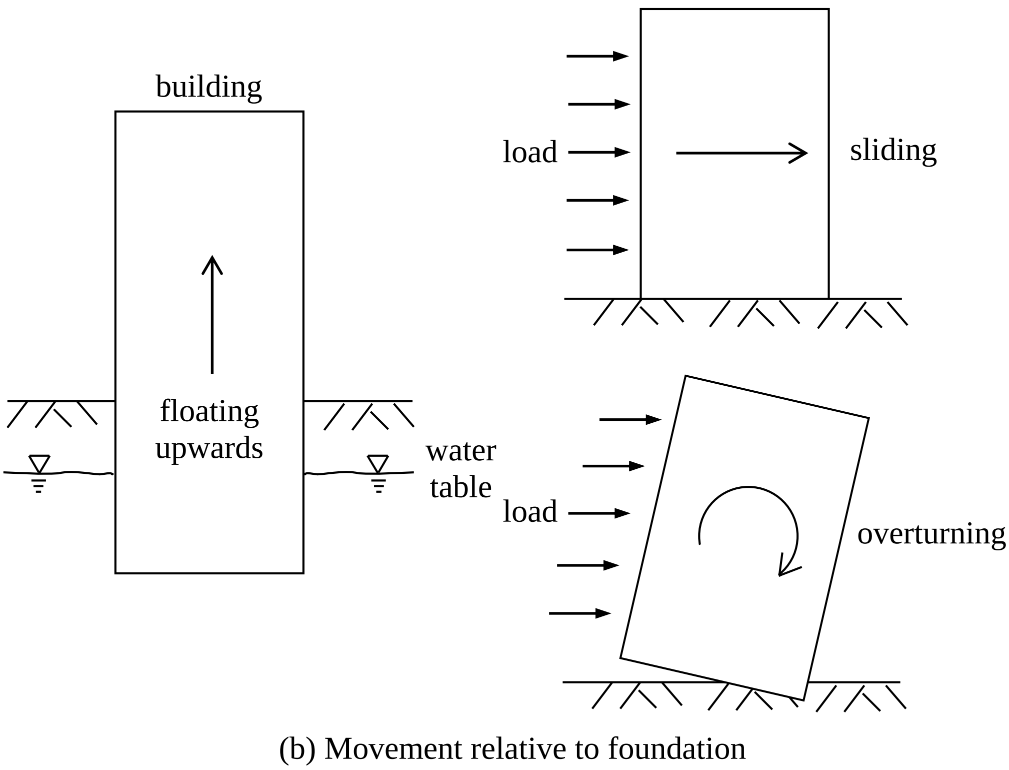
*Strength* is the primary concern, although Section 2.2 of the code touches only lightly on this aspect. When designing for strength, the design loads (with application of partial load factors to the characteristic loads) are applied to the structure and the structure is first analysed for the internal member and section forces (analysis of structure). After obtaining the internal member and section forces, each section is then analysed for the purpose of reinforcement detailing (analysis of sections). A common practice is to carry out the analysis of structure by means of linear elastic analysis and the analysis of sections by means of inelastic or plastic analysis. With this design practice, the various sections of the structure would yield at more or less the same time when the structure is loaded to the point of collapse. Such design practice is acceptable from the structural safety point of view (as the loads have been factored by partial load factors) but the resulting design may not be the most economical. Nevertheless, it has the major advantages that the design procedures are relatively simple and that computer software for linear elastic analysis of structure and inelastic/plastic analysis of sections are readily available.

If the analysis of structure is carried out by means of inelastic or plastic analysis so as to take advantage of the redistribution of internal forces from the sections that have already yielded to the other unyielded sections, a more economical design may be obtained. The method of redistribution of moments (see Section 5.2 of the code) is an abridged version of such practice. However, care is needed to check and ensure that all the yielded sections have sufficient ductility to withstand the plastic deformation so caused until the structure collapses. Ductility check is in general not easy because a rather rigorous limited ductility elasto-plastic analysis method is required. If such rigorous analysis method is not resorted to, it is better to stay with linear elastic analysis of structure and apply only those redistributions of internal forces that are explicitly permitted with clear guidelines given in Chapter 5 of the code.

*Stability* is the ability of the structure to prevent bodily movement and buckling of any part of the structure. Bodily movement is the displacement of part or whole of the structure without internal restraint (i.e. without straining any part of the structure) and without proper external restraint (i.e. without being properly affixed or restrained by external means). It is basically a first order effect. Bodily movement can occur due to formation of one or more unintentional mechanisms or due to movement of the whole structure (such as sinking, floating, sliding or overturning) relative to the foundation, as illustrated in Figure 2.1. Unintentional mechanisms could be formed in many ways, as shown in Figure 2.2. For instance, if all the shear walls of a building are arranged to be parallel to each other, the building will tend to move in the direction perpendicular to the shear walls (due to small lateral stiffness in that direction). Likewise, if all the shear walls are arranged to be intersecting at one vertical axis, the building will tend to rotate about that axis (due to small torsional stiffness). Another possibility is the connection of a cantilever beam to a shear wall at a direction perpendicular to the wall without designing for the concentrated out-of-plane bending moment that could be induced at the beam-wall joint.

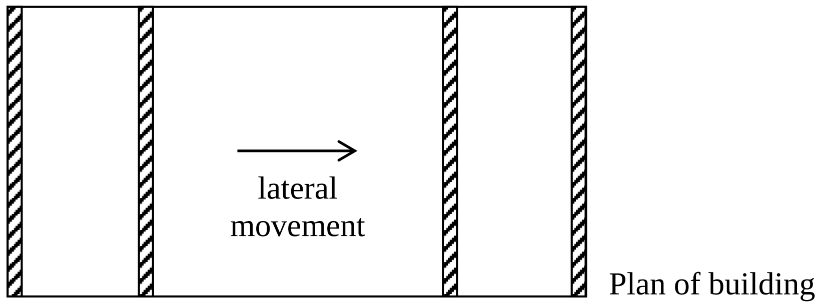



(a) A truss with an unintentional mechanism
(note that all nodes except A and H can move without internal restraint)

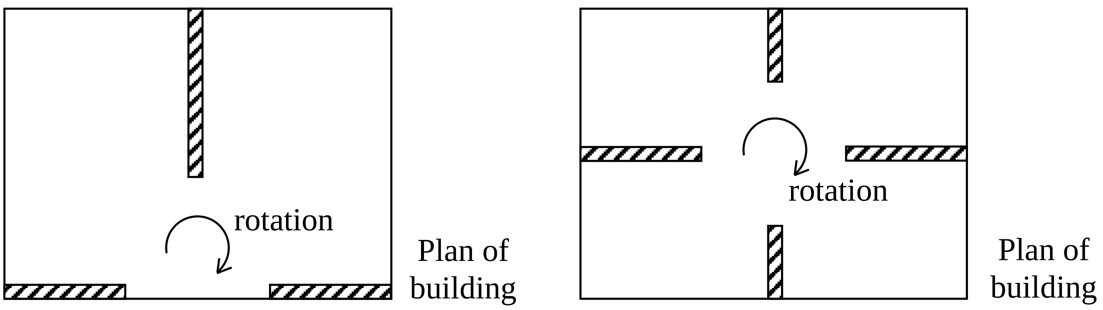



(b) Movement relative to foundation

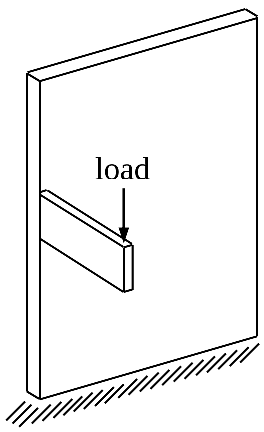
Figure 2.1 Examples of bodily movement




(a) All shear walls parallel to each other




(b) All shear walls intersecting at one point




(c) Cantilever beam connected to shear wall

Figure 2.2 Some examples of unintentional mechanisms

Buckling is the loss of stability due to geometric nonlinearity arising from the deflection of part or whole of the structure. Unlike bodily movement, buckling causes straining of the structure and is thus dependent on the stiffness of the structural elements involved (mainly the lateral stiffness of struts and out-of-plane stiffness of walls). It is basically a second order effect (also referred to as $P$-$\Delta$ effect by some engineers). The loss of stability could be gradual if the structural elements involved have initial curvature and/or eccentricity, or be very sudden with no prior warning if the structural elements are perfectly straight and in line with the applied loading. In any case, buckling is a kind of brittle failure and is therefore extremely dangerous. Buckling occurs mainly in slender structural elements. Fortunately, concrete structures are mostly quite bulky (at least compared to steel structures) and so buckling failure of concrete structures is uncommon. However, it could happen if the structure is not detailed properly. For instance, if the analysis of structure is carried out using a two-dimensional analysis method with all possible out-of-plane actions neglected and at the end no restraint against the out-of-plane movement of the structure is provided (many inexperienced engineers are tempted to forget about the possible out-of-plane actions because there is no out-of-plane loading acting on the two-dimensional model of the structure), out-of-plane buckling could occur. Excessive reliance on computer analysis may also cause the same problem because many inexperienced engineers just concentrate on structural elements subjected to loading and are not aware of the fact that sometimes structural elements have to be added to provide restraint rather than to carry loading.

*Robustness* is the fail-proof ability of the structure against disproportionate collapse when subjected to local damage due to accidental overload or impact. The aim of providing robustness is to avoid the situation whereby damage to a small part of the structure or failure of certain individual elements would lead to collapse of a large part or any major part of the structure. To provide robustness, (a) at the global scale, the general arrangement of the structure must be designed to avoid any inherent weakness that could lead to progressive collapse, and (b) at the local scale, every structural element should be designed to have at least some nominal lateral resistance and be effectively tied to other parts of the structure.

Designing the structure at the global scale to avoid inherent weakness that could lead to progressive collapse (or, in other words, designing the structure to have a good structural integrity) requires careful judgement by an experienced engineer. No simple structural analysis can replace such engineering judgement. Perhaps some kind of collapse mechanism analysis should be carried out to trace the sequence of events that could happen during collapse in order to assess the structural integrity of the structure, which is required by Clause 2.2.2.3(b) of the code. Performing such collapse mechanism analysis quantitatively is not going to be easy as the computations involved are exceedingly complicated and there is still no computer software available in the market, although research has pointed out the possibility of applying nonlinear finite element analysis for this purpose. Anyhow, at the least, conceptual and qualitative collapse mechanism analysis should be carried out to identify any key structural elements, the failure of which would cause the collapse of more than a limited portion close to the element in question. As a general rule, as far as practicable, the arrangement of the structure should be designed such that only a small number of structural elements are indispensable (these are called key elements) and the indispensable elements (the key elements) must be designed to resist accidental loading and/or protected to prevent removal by accident. Another general rule is to provide as much redundancy as possible so that when a certain part of the structure fails, the loading that it is carrying may be redistributed for eventual transmission to the foundation without causing progressive collapse.

At the local scale, every structural element should be designed to have some nominal resistance in all directions at least to the ultimate lateral loads and/or the code specified notional loads. For instance, a beam subjected mainly to vertical downward loading and sagging moment should also be designed to be capable of resisting certain nominal vertical upward loading and hogging moment because the direction of accidental loading is in general very unpredictable and can act in any direction. When there are elements that are effective in resisting load in only one direction, such as guy wires, stay cables and suspension cables, particular care is needed to ensure that when the loading direction reverses, the structure still has sufficient resistance to avoid initiation of progressive collapse. In addition, every structural element should be properly connected to other parts of the structure. For connecting precast components to the structure, monolithic connections are generally preferred. If practicable, the reinforcing bars to be anchored at the connections should be mechanically connected or welded together instead of just lapped side by side to improve the structural integrity.

Following the events of September 11 in 2001, much research has been carried out into the robustness of structures particularly with respect to disproportionate collapse of highrise buildings. The two principal reports relating to this topic are "Safety in Tall Buildings" published by the UK Institution of Structural Engineers and "World Trade Center Building Performance Study" published by the USA Federal Emergency Management Agency. Their recommendations essentially follow the guidelines given in BS8110 and Eurocode 2, namely:

- Identifying any key elements in each structure, for which their failure would lead to the collapse of a greater portion of the building than that local to the element in question. Where these key elements cannot be designed out, the design of these elements will take their importance into account using appropriate measures.
- Ensuring the structures will be capable of safely resisting a notional horizontal design ultimate load at each floor level.
- Ensuring the structures will be provided with effective horizontal ties around their peripheries, internally and to their vertical elements.
- Ensuring that the vertical elements of the structure will be provided with ties such that the removal of a vertical element, other than a key element, will result in any collapse being limited to the portion of the structure in close proximity to that element.

These guidelines have been incorporated into the code of practice.

Handbook to British Standard BS 8110:1985 states two basic principles about the general check of structural integrity:

1. The overall form of the structure should be chosen so that it is not excessively flexible to any mode of deformation;
2. The form of the structure should be such that the centre of resistance of the structure to a particular loading should be close to the line of action of the loading.

For very special and important buildings, it may be necessary to allow for the effects of particular hazards or for an unusually high probability of the structure surviving an accident even though damaged. In such case, safety factors higher than those given in the code may be required.

In Hong Kong, structures are normally designed with sufficient stability and robustness so that the effects of impact and impassive loading should be small. The danger against sabotage and terrorist attacks is not considered high in general. Useful information on protective system against these loadings can be found in CEB and fib publications. If building structures are required to be designed against these extreme actions, specialist literature should be consulted and the design carried out accordingly.

For easy reference, a flowchart of the robustness design procedure, similar to the one given in BS8110: Part 1: 1997, is presented in Figure 2.3.

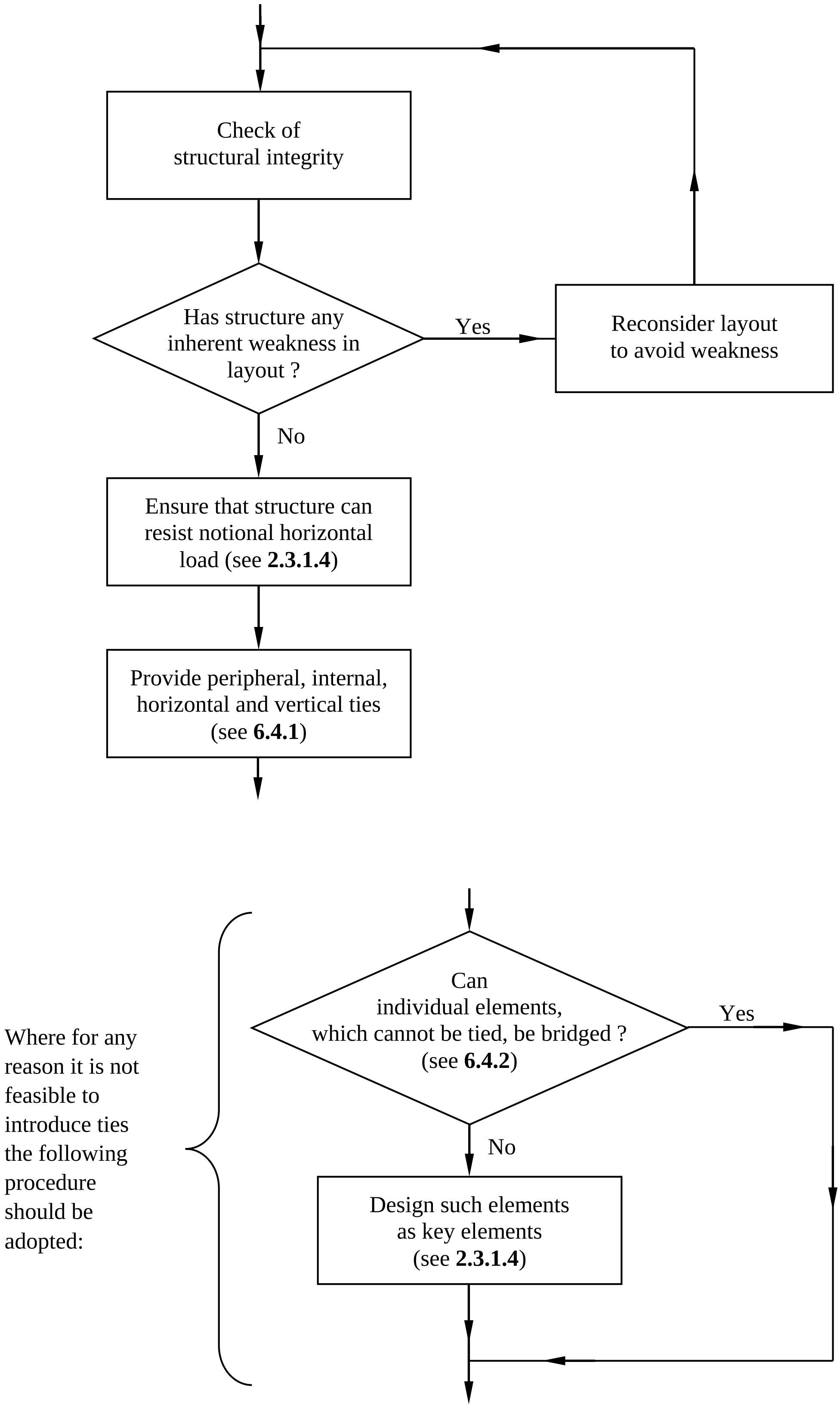



Figure 2.3 Flowchart of robustness design procedure
(the clause numbers refer to those in the code)

2.2.3 Fire limit state

The provisions relating to fire limit state in this Code aimed to provide guidance for checking the structural adequacy of reinforced concrete members at elevated temperature due to fires. Whereas, the assessment of fire loads, analysis for a design fire, etc. are outside the scope of this Code.

2.2.4 Serviceability limit state (SLS)

SLS design is concerned mainly with the functionality of the structure (depending on the type and intended usage of the structure, functionality here may include fitness for purpose, user comfort, appearance and other special requirements such as water-tightness of the structure). To satisfy the SLS requirements, the *cracking, deflection* and *vibration* of the structure under normal usage are analysed and checked against the acceptable limits stipulated in the code of practice and if applicable additional requirements set by the client. Traditionally, the *fatigue resistance, durability* and *fire resistance* of the structure are also evaluated and checked as a part of the SLS design, although these performance requirements should have been designed for in a separate design routine (fatigue resistance, durability and fire resistance are not really subsets of serviceability).

The analysis of structure for evaluation of cracking, deflection and vibration is usually carried out by means of linear elastic analysis because the structure is expected to remain elastic under normal usage. In many cases, where the cracking, deflection or vibration are not expected to cause concern, deemed-to-satisfy design rules for dimensioning and detailing of the structural elements (e.g. maximum span/depth ratios, minimum reinforcement areas and maximum steel bar spacings) have been provided so as to simplify the design process.

*Cracking* affects the appearance, durability and water-tightness of the structure. Broadly speaking, there are two categories of cracks: non-structural cracks (also called restraint-induced cracks) and structural cracks (also called load-induced cracks). Non-structural cracks are due to sedimentation, shrinkage or thermal movement of the concrete, which if restrained, would induce tensile stresses large enough to cause cracking. Structural cracks are due to tensile stresses developed in the concrete resulting from the applied loads. Non-structural cracks may be controlled by proper concrete mix design, careful planning of the construction sequence, proper control of the temperature and moisture conditions during curing, provision of movement joints and provision of crack distribution reinforcement. On the other hand, structural cracks may be controlled by limiting the tensile stresses developed in the concrete, proper reinforcement detailing and provision of crack distribution reinforcement.

The necessity to limit the *deflection* arises from the problem that excessive deflection of the structure could lead to drainage difficulties, damage of the non-structures such as finishes, partitions, glazing, cladding and services (these normally could not accommodate too much deflection), and movement or vibration that could affect the user comfort of the structure.

*Vibration* is of concern because it may cause discomfort to the occupants and/or interference with proper function of sensitive equipment housed in the building. There are three major sources of vibration: wind-induced vibration, machine-induced vibration and footfall vibration. Wind-induced vibration may be limited by controlling the lateral deflection of the building under the equivalent static wind load or by controlling the peak acceleration of the building under the dynamic wind load. On the other hand, machine-induced vibration should better be controlled by isolating the source of vibration. For footfall vibration, the magnitude is normally small but it may impair the function of sensitive equipment. Footfall vibration can be controlled by stiffening structural elements or installation of dampers.

## 2.3 Loads

### 2.3.1 Design loads

The loads are defined in terms of characteristic loads, which should have a reasonably high probability of not being exceeded under normal usage during the design working life of the building. The characteristic dead load, imposed load and wind load are given in the Code of Practice for Dead and Imposed Loads for Buildings and the Code of Practice on Wind Effects.

The design load, i.e. the load value to be used in the design calculations, is to be taken as the characteristic load multiplied by an appropriate partial safety factor (i.e. load safety factor), which is dependent on the type of loading and limit state.

A set of design loads for robustness design has, for the first time, been given in the Hong Kong codes. The design loads for robustness (no partial safety factor is to be applied) comprise of:

(a) A notional ultimate horizontal load at each and every floor level equal to 1.5% of the characteristic dead weight of the structure between mid-height of the storey below and mid-height of the storey above.
(b) A notional ultimate load of 34 $kN/m^2$ on each key structural element, acting in any direction (vertical or horizontal, upward or downward, and inward or outward). The projected area of the key element is to be used for the determination of the ultimate load.
(c) A notional ultimate load of 34 $kN/m^2$ on each building component connected to a key structural element acting in any direction. The projected area of the building component is to be used for the determination of the ultimate load but the reaction to be transmitted to the key structural element should be the maximum that might reasonably be transmitted having regard to the strength of the component and the strength of the connection.

Following the progressive collapse of Ronan Point in 1968 (a UK residential building that suffered extensive damage following a gas explosion in one of the apartments), regulatory requirements were introduced in UK to provide (in buildings above 5 storeys) structural resistance with the aim of limiting damage caused by an accident or misuse so that it is not disproportionate to the cause. These rules are included in both the UK Building Regulations and the British Standard code of practice. The value of 34 kN/m$^2$ originates from research carried out by the Building Research Establishment following the gas explosion at Ronan Point. Measurements of peak pressure following such incidents were assessed and the value of 34 kN/m$^2$ was found to be the peak pressure that was exceeded by only 2% of incidents. It is therefore of the order of a 1 in 50 event. This was considered to be a reasonable value to use in design.

The notional ultimate horizontal load in (a) above should not in general govern the design of buildings in Hong Kong, because of the relatively large wind loads to be designed for. What needs to be done during the design process is just to make sure that the design ultimate wind load to be applied at each floor level is not smaller than this notional ultimate horizontal load for robustness.

In most cases, the notional ultimate loads in (b) and (c) above should have little effect on the design of the building structure, but still design check is required to confirm compliance with these two robustness requirements. Perhaps the greatest effect is on the provision of out-of-plane resistance to each shear wall for robustness. The projected area of a shear wall in the out-of-plane direction is quite large. This, multiplied by the notional ultimate load of 34 kN/m$^2$, could lead to a fairly large loading. Even then, the robustness requirement would govern the design of the shear wall only when the thickness of the wall is relatively small.

### 2.3.2 Loads for ultimate limit state

The major loads to be considered for ULS design are dead load, imposed load and wind load, which are to be combined into at least three different load combinations: (dead load + imposed load), (dead load + wind load), (dead load + imposed load + wind load), as per Table 2.1 of the code. It should be noted that although Table 2.1 resembles the table in Clause 2.4.3 of BS8110: Part 1: 1997, the load safety factors given in Table 2.1 are not the same as those given in the British Standard. In BS8110, the beneficial load safety factor for imposed load in the (dead load + imposed load + wind load) combination has been set as 1.2. This is actually wrong because the imposed load may be present or absent, and when the presence of imposed load is beneficial, the absence of imposed load would produce a more critical condition. Therefore, the beneficial load safety factor should have been taken as 0. In the code 2004 and onward, this mistake is corrected and the beneficial load safety factor for imposed load is set equal to 0.

The code also suggests the loads factors for dead load, imposed load and wind load under fire limit state, as per Table 2.2, which tally with the load factors stated in Table 12.5 of Code of Practice for Structural Use of Steel 2011.

### 2.3.3 Loads for serviceability limit state

The major loads to be considered for SLS design are dead load, imposed load, wind load, differential settlement of foundation (if any), and creep, shrinkage and temperature effects (if any). For SLS design, the load safety factors for all types of loading are taken as 1.0. In most cases, if the design and detailing of the structural elements are carried out in accordance with the deemed-to-satisfy rules given in the code, no further checks on SLS are required.

## 2.4 Materials

The strength of a material is defined in terms of its characteristic strength, which should have a reasonably high probability of being achieved with proper quality control and site supervision assumed. The design strength, i.e. the strength value to be used in the design calculations, is to be taken as the characteristic strength divided by an appropriate partial safety factor (i.e. material safety factor), which is dependent on the variability of the strength parameter, possible difference between the in-situ strength and the laboratory measured strength value, resulting uncertainty in the estimated sectional resistance and limit state.

In the older 1985 edition of BS8110, the material safety factor $\gamma_m$ for steel reinforcement is taken as 1.15, while in the newer 1997 edition of BS8110, the material safety factor $\gamma_m$ for steel reinforcement is changed to 1.05. Both these two material safety factors have been used in Hong Kong. However, bearing in mind that the sources of reinforcement in Hong Kong are quite diversified with varying degrees of quality control, the reduction of the material safety factor for reinforcement from 1.15 to 1.05 is not considered advisable, at least at this stage. Hence, the material safety factor for reinforcement of 1.15 in the 1985 edition of BS8110 is adopted.

The Code includes partial safety factors for material strength in fire limit state, as per Table 2.3, which tally with the material factors stated in Table 12.4 of Code of Practice for Structural Use of Steel 2011.

## 2.5 Analysis and verification

As stated before, the analysis for the purpose of reinforcement detailing and verification of compliance with all the limit states consists of two stages:

- analysis of structure; and
- analysis of sections.

Guidelines for the analysis of structure and analysis of sections are given in Chapters 5 to 7 of the code. However, these rules are limited to design of structures under static loads or equivalent quasi-static loads of wind. For dynamic analysis, specialist guidance should be sought. Where non-linear effects, such as those due to geometric nonlinearity or material nonlinearity, are significant, specialist guidance should also be sought. Readers may refer to a reference for details: M.A.N. Hendriks, A. de Boer, B. Belletti, Guidelines for Nonlinear Finite Element Analysis of Concrete Structures, Rijkswaterstaat Centre for Infrastructure, Report RTD:1016-1:2017, 2017, 69pp.

For analysis of column and wall, Eurocode 2 recommended an eccentric value of $e_i = l_o/400$, where $e_i$ is the amplitude of the initial bow imperfection assumed as a half-sine function and $l_o$ is the initial member length, to cover imperfections if nonlinear analysis is to be carried out as perfect structures without imperfection is unsafe. Consideration of imperfections is important especially for those made of high strength materials, e.g. high strength concrete, due to greater design strength to stiffness ratio.

## 2.6 New and alternative methods

New and alternative design methods not found in the code may also be used provided it can be demonstrated that the basic performance requirements in Section 2.1 of the code are complied with and that the overall standards of the design are not inferior compared to the general standards stipulated in the code. For such purpose, the design engineer may have to verify the accuracy of the theoretical analysis by model and prototype tests and provide evidence that all the performance requirements have actually been achieved by field monitoring and/or testing.

# 3 MATERIALS

## 3.1 Concrete

General

This section applies only to concrete made from the locally available rock aggregate (mainly granite rock aggregate).

Unlike the British Standard BS8110, which covers only normal-strength concrete up to grade C60, high-strength concrete up to grade C100 is covered in the new code. This represents a great advancement as the usage of high-strength concrete is increasing and more design guidance is desperately needed.

The properties of concrete are dependent on the raw materials (mainly the type of aggregate used), the curing conditions and the local concreting practices. Hence, the characteristics of concrete made in different places are not quite the same and the material properties of concrete given in other codes are not necessarily applicable to Hong Kong. To resolve this problem, the local universities have for many years conducted research on the properties of the local concrete. It has been found that generally speaking the local concrete made with granite rock aggregate has lower elastic modulus, lower strength with the same mix proportion and larger shrinkage compared to most other concretes made elsewhere. These characteristics of the local concrete have already been incorporated in the new code.

The recommendations made in this section on the properties of the local concrete and on the usage of high-strength concrete are based mainly on the research studies carried out at The University of Hong Kong.

Elastic deformation

The elastic modulus of concrete is given in the new code in both an equation form and a table form. The equation for the evaluation of elastic modulus is:

$$E_c = 3.46\sqrt{f_{cu}} + 3.21 \qquad \text{Equation 3.1}$$

where $E_c$ is the short-term elastic modulus of the concrete (in kN/mm$^2$) and $f_{cu}$ is the cube compressive strength (in N/mm$^2$). It is applicable to concrete with cube strength ranging from 20 to 100 N/mm$^2$.

The elastic modulus values evaluated as per the above equation agree closely with the corresponding values given in the previous Hong Kong Building Code – Buildings and Lands Department, Code of Practice for the Structural Use of Concrete: 1987 within the cube strength range of 20 – 45 N/mm$^2$ and the Hong Kong Highways Code – Highways Department, Structures Design Manual for

Highways and Railways: 1997 within the cube strength range of 20 – 60 N/mm$^2$. Hence, up to a cube strength of 60 N/mm$^2$, there is little change in the elastic modulus. The major change in the new code is the extension to cover cube strengths from 60 to 100 N/mm$^2$.

It should be noted that when checking the overall building deflection or relative lateral deflection at the transfer structure level, the mean elastic modulus may be used. The mean elastic modulus are obtained by substituting the mean concrete strength in the equation mentioned above, while the mean concrete strength is at least 5MPa greater than the characteristic concrete strength as specified in B(C)R. The mean strength is, therefore, adopted as the characteristic concrete strength plus 5 MPa.

It should be noted that when the characteristic elastic modulus is required, the characteristic cube strength should be used instead.

The elastic modulus of the local concrete as given in the new code is based on extensive tests carried out at The University of Hong Kong, which have been published in the following paper:
Kwan A.K.H., Zheng W. and Lee P.K.K., "Elastic modulus of normal- and high-strength concrete in Hong Kong", Transactions, Hong Kong Institution of Engineers, Vol.8, No.2, 2001, pp10-15.

Creep and shrinkage

The methods of estimating the creep and shrinkage of concrete in the code are similar as those given in the Hong Kong Highways Code – Highways Department, Structures Design Manual for Highways and Railways: 1997, which are actually based on Appendix C of BS5400: Part 4: 1990. According to BS5400: Part 4, the creep and shrinkage of concrete may be evaluated using the following equations:

$$\emptyset_c(t) = K_L K_m K_c K_e K_j \qquad \text{Equation 3.3}$$
$$\varepsilon_{cs}(t) = K_L K_c K_e K_j$$

in which $\emptyset_c$ is the creep coefficient, $\varepsilon_{cs}$ is the shrinkage strain, and the "$K$" coefficients are as defined in BS5400: Part 4. However, over the years, it had been found from field measurements and laboratory tests that the shrinkage of the local concrete is generally much larger than that predicted by the BS5400: Part 4. The most probable cause of the much larger shrinkage of the local concrete may be attributed to the properties of the granite rock aggregate being used in Hong Kong. In order to allow for the larger shrinkage of the local concrete, the Hong Kong Highways Code incorporates a modification factor $c_s$ into the formula for shrinkage, as depicted below:

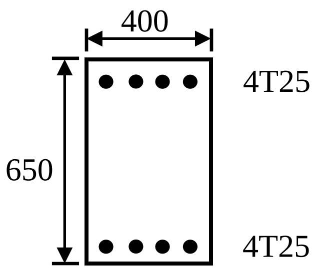
$$\varepsilon_{cs}(t) = c_s K_L K_c K_e K_j \qquad \text{Equation 3.5}$$

The modification factor $c_s$ has been assigned a value of 2.3 in the Highways Department, Structural Design Manual for Highways and Railways.

The modification factor of 2.5 implies that the shrinkage of the local concrete is about 2.5 times that of similar concrete in UK. Such implication has aroused the controversy whether the shrinkage of concrete in one place can be 2.5 times that of similar concrete in another place, despite the fact that the shrinkage of concrete is heavily influenced by the properties of the constituent materials. The modification factor $c_s$ of 2.3 or 2.5 both reflected the shrinkage properties of the local concrete. The University of Hong Kong conducted the research by developing a shrinkage model and revealed that early experimental studies in Hong Kong suggested that the shrinkage of Hong Kong concrete is considerably larger; which has been published in the following paper:
Kwan A.K.H., Au F.T.K., Wong H.H.C. and Ng P.L., "Shrinkage of Hong Kong granite aggregate", Magazine of Concrete Research, Vol. 62, No. 2, 2010.

<u>Worked example 3.1: Creep and shrinkage strain</u>

Calculate the creep strain and shrinkage strain at top fiber of the following beam under no lateral restraint, as shown in Figure 3.1, which subject to bending moment at 100kN-m.




Properties of the beam:

A = 260000 mm$^2$
I = 9.15 × 10$^9$ mm$^4$

Figure 3.1 Worked example - creep strain and shrinkage strain determination

Data: $A_s$ = 3920 mm$^2$, Concrete grade 40, $E_c$ = 25100 N/mm$^2$

<u>For Creep Strain:</u>

$K_L$ = 2.3 (Normal air) Figure 3.1

$K_m$ = 1.0 (Ordinary Portland cement, 28-day) Figure 3.2

$K_c$ = 0.8 (water-cement ratio = 0.4 and cement content = 400 kg/m$^3$) Figure 3.3

$u$/2 (semi-perimeter in contact with atmosphere) $= {}^{2(400+650)}/_{2}$

$= 1050$ mm

$h_e$ (ratio of area of session to $u$/2) $= {}^{260000}/_{1050} = 248$ mm

$K_e$ = 0.8 Figure 3.4

$K_j$ = 1.0 (for long-term) Figure 3.5

$\alpha_e = {}^{200000}/_{25100} = 7.97$

$\rho = {}^{3920}/_{260000} = 0.015$

$K_s = {}^{1}/_{(1 + 0.015 \times 7.97)} = 0.89$ Equation 3.4

$\emptyset = K_L K_m K_c K_e K_j K_s = 2.3 \times 1.0 \times 0.8 \times 0.8 \times 1.0 \times 0.89 = 1.31$ Equation 3.3

$f_c$ (bending stress at extreme fibre, assuming elastic and non-cracked section) $= \left({}^{100 \times 10^6}/_{9.15 \times 10^9}\right)(325) = 3.55\ \text{N/mm}^2$

$\varepsilon_{cc} = 1.31 \times {}^{3.55}/_{25100} = 1.85 \times 10^{-4}$ Equation 3.2

<u>For Shrinkage Strain:</u>

$c_s = 2.5$

$K_L = 275 \times 10^{-6}$ (Normal air) Figure 3.6

$K_c = 0.8$ Figure 3.3

$K_e = 0.73$ Figure 3.7

$K_j = 1.0$ Figure 3.5

$K_s = 0.89$

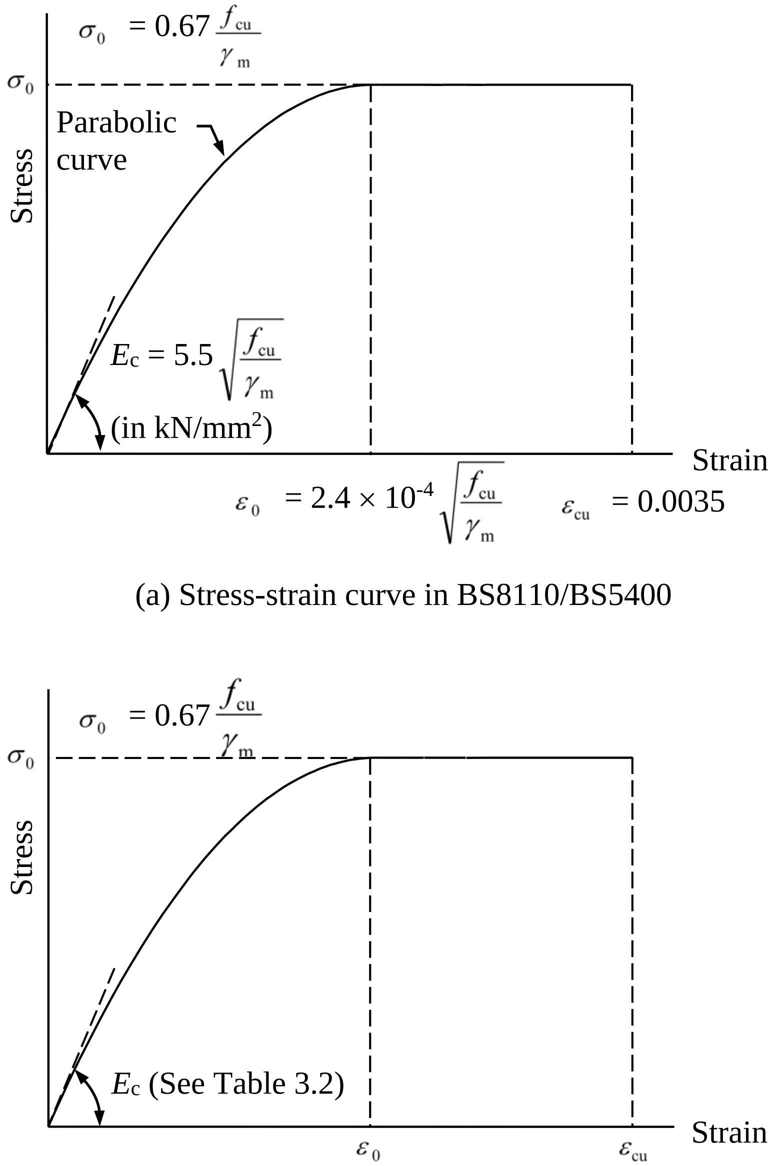
$\varepsilon_{cs} = c_s K_L K_c K_e K_j = 2.5 \times 275 \times 10^{-6} \times 0.8 \times 0.73 \times 1.0 \times 0.89$

$= 3.573 \times 10^{-4}$ Equation 3.5

Note: All figure number and equation number in the worked example are referring to those in Code of Practice for Structural Use of Concrete 2013.

Stress-strain relationships for design

The stress-strain curve of concrete given in the new code is compared to that of BS8110/BS5400 in Figure 3.2.




(a) Stress-strain curve in BS8110/BS5400



(b) Stress-strain curve in the new code

Figure 3.2 Stress-strain curves of concrete

The stress-strain curve in the new code differs from that of BS8110/BS5400 in the following two ways:

(a) The initial gradient of the curve (i.e. $E_c$) has been changed to match with the generally lower elastic modulus of the local concrete.

(b) The reduction of the ultimate concrete strain $\varepsilon_{cs}$ (the strain at the extreme compression fibre when the concrete fails) with the increase of the concrete strength $f_{cu}$ to beyond 60 N/mm$^2$ has been taken into account so as to extend the applicability of the curve to high-strength concrete.

However, although Table 3.2 has been referred to for the evaluation of the initial gradient $E_c$, the method of applying the material safety factor $\gamma_m$ has not been specified. In theory, the material safety factor should be applied before evaluating the initial gradient using either Table 3.2 or Equation 3.1 of the code. Since after application of the material safety factor, the concrete strength becomes ${f_{cu}}/{\gamma_m}$ and is no longer a whole number, it should be better to use Equation 3.1, which after the material safety factor is applied, becomes:

$$E_c = 3.46\sqrt{\left({f_{cu}}/{\gamma_m}\right)} + 3.21 \qquad \text{Equation 3.1}$$

After changing the initial gradient $E_c$, the value of $\varepsilon_0$ needs to be adjusted. Mathematically, for the initial part of the stress-strain curve to remain a parabolic curve, the value of $\varepsilon_0$ needs to be set equal to ${2\sigma_0}/{E_c}$. Hence, for mathematical compatibility, the value of $\varepsilon_0$ should have been adjusted to:

$$\varepsilon_0 = \frac{1.34(f_{cu}/\gamma_m)}{E_c} \qquad \text{Figure 3.8}$$

which is in general larger than that of BS8110/BS5400.

The value of the ultimate concrete strain $\varepsilon_{cu}$ has to be reduced when the concrete strength $f_{cu}$ is higher than 60 N/mm$^2$ because high-strength concrete generally has a stress-strain curve that would, after reaching the peak, decline more rapidly than in normal-strength concrete (high-strength concrete is more brittle). To allow for such effect, the code has provided the following formula for the evaluation of $\varepsilon_{cu}$:

$$\varepsilon_{cu} = 0.0035 \text{ for } f_{cu} \leq 60 \text{ N/mm}^2 \qquad \text{Figure 3.8}$$

$$\varepsilon_{cu} = 0.0035 - 0.00006\sqrt{(f_{cu} - 60)} \text{ for } f_{cu} > 60 \text{ N/mm}^2 \qquad \text{Figure 3.8}$$

This formula is based on the research study carried out at The University of Hong Kong, which has been published in the following paper:

Ho J.C.M., Kwan A.K.H. and Pam H.J., "Ultimate concrete strain and equivalent rectangular stress block for design of high-strength concrete beams", Structural Engineer, Vol.80, No.16, 2002, pp26-32.

## 3.2 Reinforcing steel

The grades of steel and their mechanical properties given in the code are revised from High yield steel, 460N/mm$^2$ to Grade 500B and 500C, 500N/mm$^2$.

Reinforcing bar couplers are proprietary products designed and manufactured to comply with the relevant design code or an alternative standard accepted by the Building Authority. Apart from satisfying certain strength requirements, the coupled bar assembly should also comply with certain requirements in respect of deformation characteristics. It is often expected that the structural performance of a concrete member with coupled bar assemblies is not inferior to that with the equivalent continuous bars in all aspects.

The cyclic tension-and-compression test of type 2 mechanical coupler is to evaluate the performance of the splice reinforcement following significant inelastic stress reversal fatigue cycles. The test initiates with a series of elastic stress reversal cycles and increasing inelastic stress reversal cycles up to 5 times the yield strain of the reinforcement being tested and load to failure in the last stage.

## 3.3 Welded fabric

The requirements of welded fabric are stated in this section. The material properties of Grade 500A steel reinforcement should comply with BS4449. For those of Grade 500B and Grade 500C, CS2 should be complied with. The fabrication, sampling and testing other than material properties should comply with BS4483.

## 3.4 Prestressing tendons

The stress-strain relation and other mechanical properties given in the code are basically the same as those in BS8110. It should be noted that the 0.5% proof stress $f_{pu}$ is used in place of the yield stress $f_y$ (which may not be well defined) to define the ultimate strength of prestressing tendons.

## 3.5 Prestressing devices

The requirements for prestressing devices are given in this section in a very brief manner because no general rules can yet be explicitly and exhaustively set. It is better to either employ proprietary products with good performance records or employ only those new products with sufficient field trials carried out to demonstrate their compliance with all the performance requirements in Section 2.1 of the code.

## 3.6 New materials

New materials not covered in the code may also be used provided it can be demonstrated that the basic performance requirements in Section 2.1 of the code are complied with and that the overall standards of the materials are not inferior compared to the general standards stipulated in the code. For such purpose, the design engineer has to provide sufficient information, including manufacturing data, track record of previous applications, testing and proposed quality controls, to allow independent third party evaluation.

## 3.7 Design strength at elevated temperatures

The reduction factor to design strength of concrete, reinforcement and prestressing tendon at elevated temperature are stated in Tables 3.5, 3.6 & 3.7, which are retrieved from Figure 4.1 (curve 1), 5.1 (curve 1) and 5.1 (curves 2 & 3) of EC 2 Part 1-2.

# 4 DURABILITY AND FIRE RESISTANCE

## 4.1 Objectives

### 4.1.1 Durability

The purpose of durability design is to ensure that the structure will perform its intended functions satisfactorily for a sufficiently long period of time or at least during its design working life without requiring excessive maintenance. Durability design requires consideration right at the beginning of the design stage, although some engineers tend to leave this important issue to the stage of writing up the specification.

Durability design is not just a material problem. Crack control is at least as important. No matter how good the materials are, if the structure cracks extensively to allow direct ingress of the aggressive ingredients into the core of the structure, the structure will not last long. There are two types of cracks: non-structural cracks and structural cracks. They could be caused by use of an inappropriate concrete mix, inadequate temperature/moisture control during curing, restraint against shrinkage/temperature movement and excessive tensile strains induced by the applied load. Hence, as the code says, it is dependent upon the integration of every aspect of design, materials and construction.

The guidelines for durability design provided in the code are applicable only to structures in normal environment and with a design working life of 50 years. For more severe environment and/or design working life longer than 50 years, additional protection measures, such as the incorporation of cathodic protection, protective coatings and corrosion inhibitors etc, which have not been included in the code, may be necessary.

As part of the durability design, it is strongly advised to carry out life-cycle cost analysis of the structure, taking into account the initial cost of construction, the future cost of maintenance and the consequential loss of utility due to disruptions caused by the maintenance works required. Life-cycle cost analysis is not yet common in Hong Kong but is already well established in many other places as a useful tool for deciding the level of protection to be provided.

### 4.1.2 Fire resistance

The purpose of fire resistance design is to ensure that every structural element would possess an appropriate degree of resistance to flame penetration, heat transmission and collapse during fire attack.

From literature review, it would appear that the most common approach to fire engineering is to use the prescriptive method. Each regional respective code gives requirements for minimum cover and minimum member size based on research on the characteristic behaviour of that particular regional construction materials, with respect to concrete and reinforcement strengths and types of aggregate etc. Hong Kong already has guidance on cover requirements as published in the Code of Practice for Fire Resisting Construction. The guidance is based on the local conditions.

As with the prescriptive approach, the fire engineering calculation method in each regional code is based on calibrated testing specific to that region. It is therefore questionable whether the procedures and formulae specified in either the Eurocode or BS8110 are applicable to Hong Kong. In addition, the accuracy of the data on which the calculations are based would also be called into question as it represents a notional fire development rate used for the standardised testing of materials. In practice, a real fire may follow a significantly different pattern depending on the quantity and type of combustible materials present. Therefore, in the absence of any clear benefit, it is considered not appropriate to include the calculation method in the Hong Kong Code of Practice until further Hong Kong specific research has been carried out.

## 4.2 Requirements for durability

### 4.2.1 General

A reinforced concrete structure may deteriorate because of deterioration of the concrete itself or because of corrosion of the steel reinforcing bars inside the concrete. Common causes of deterioration of concrete include alkali-aggregate reaction, chemical attack, freezing and thawing action and mechanical abrasion. In most cases, it is the corrosion of the steel reinforcing bars that is more likely to be the major problem.

The corrosion of steel is an electro-chemical reaction. When there exists a difference in electrical potential along the steel bar in concrete, an electro-chemical cell is set up: there form anodic and cathodic regions, connected by an electrolyte in the form of pore water in the hardened cement paste. The positively charged ferrous ions $Fe^{++}$ at the anode pass into the pore solution while the negatively charged free electron $e^-$ pass through the steel into the cathode where they are absorbed by the constituents of the electrolyte and combine with water and oxygen to form hydroxyl ions $(OH)^-$. The hydroxyl ions travel through the electrolyte and combine with the ferrous ions to form ferrous hydroxide, which is converted by further oxidation to rust. The electro-chemical reactions involved are as follows:

*Anodic reactions*:

$Fe \rightarrow Fe^{++} + 2\,e^-$

$Fe^{++} + 2\,(OH)^- \rightarrow Fe(OH)_2$ (ferrous hydroxide)

$4\,Fe(OH)_2 + 2\,H_2O + O_2 \rightarrow 4\,Fe(OH)_3$ (ferric hydroxide)

*Cathodic reaction*:

$4\,e^- + O_2 + 2\,H_2O \rightarrow 4\,(OH)^-$

Both water and oxygen (or air) are needed for the process to continue. There is no corrosion in dry concrete (because there is no water) nor in concrete fully immersed in water (because there is little oxygen or air). The optimum relative humidity for corrosion is 70 to 80%.

The differences in electro-chemical potential can arise from differences in the environment of the concrete, for example when a part of it is wet and another part is dry. A similar situation can arise when there is a substantial difference in the thickness of cover to the steel.

Fortunately, even with a continuous supply of oxygen and water, the steel in concrete does not necessarily corrode. The concrete cover protects the steel from corrosion not just by hindering the ingress of deleterious fluids but also by means of *passivation*. Steel embedded in hydrating cement paste rapidly forms a thin passivating layer of oxide, which strongly adheres to the underlying steel and gives it complete protection from reaction with oxygen and water, i.e., from corrosion. This state of the steel is known as passivation. Maintenance of passivation is conditional on an adequately high pH or in other words high alkalinity of the pore water in contact with the passivating layer. Basically, concrete is alkaline because of the presence of lime, i.e. $Ca(OH)_2$, which is liberated as a byproduct during cement hydration. The pH of the pore water in hardened Portland cement paste is initially around 12.6 to 13.5, which is high enough to offer passivation protection to the steel.

However, there is everywhere carbon dioxide, i.e. $CO_2$, in the air. Carbon dioxide reacts with moisture to form carbonic acid, which then reacts with the lime in the pore water of concrete to form calcium carbonate, a neutral product. This is called carbonation. As a result, the alkalinity of the concrete gradually drops and once the pH is reduced to below around 10, the passivation protection to the steel will be gone and if there is oxygen and water, the steel will start to corrode.

Apart from carbon dioxide, chloride ions may also destroy the protective passivity layer on the surface of embedded steel thereby causing de-passivation. Chloride ions activate the surface of the steel to form an anode, the passivated surface being the cathode. The reactions involved are as follows:

$$Fe^{++} + 2\,Cl^{-} \rightarrow FeCl_2 \quad \text{(ferrous chloride)}$$
$$FeCl_2 + 2\,H_2O \rightarrow Fe(OH)_2 + 2\,HCl$$

Thus, the chloride ion $Cl^{-}$ is regenerated so that the rust contains no chloride, although ferrous chloride is formed at the intermediate stage.

Rust has a lower density than steel. Hence, as corrosion takes place, the volume increases and since the expansion is restrained by the surrounding concrete, bursting stresses are induced, resulting in cracking, spalling or delamination of the concrete cover. This makes it easier for aggressive agents to seep through the concrete towards the steel, with a subsequent increase in the rate of corrosion. In other words, once corrosion starts, it will accelerate.

With the exception of mechanical damage, all the adverse influences on the durability of concrete involve the transport of fluids or ions through the concrete. There are four fluids/ions principally relevant to the durability of concrete: water, oxygen, carbon dioxide and chloride ions. They can move through concrete in different ways:

- *permeation* (flow under pressure gradient);
- *diffusion* (flow under concentration gradient);
- *sorption* (flow under capillary suction).

As sorption is normally insignificant, the major factors influencing the durability are the permeability and diffusivity of the concrete.

Permeation and diffusion are dependent on the size, porosity and connectivity of the pores. Since the same geometric factors influence both permeation and diffusion, permeability and diffusivity are inter-related and it may be said that when the permeability is high the diffusivity is also likely to be high and vice versa. Test results have shown that they are roughly proportional to each other and consequently they are often loosely treated as equivalent (the common practice of referring to the permeability of concrete to carbon dioxide or chloride ions is actually wrong because it is the concentration gradient, not the pressure gradient, that drives the carbon dioxide or chloride ions through the concrete).

However, unlike permeation, the diffusion of gases or ions through concrete or any porous medium is sensitive to the relative humidity or the degree of saturation of the pores. The diffusion of gases, such as oxygen and carbon dioxide, through dry pores is much faster than through wet pores or through the pore water. In contrast, the diffusion of ions, such as chlorides and sulphates, takes place only through the pore water and thus ionic diffusion is possible only when the pores are saturated or at least partially saturated.

There are different types of pores in concrete: gel pores, capillary pores and air voids in the hardened cement paste and pores in the rock aggregate, each of different size and therefore contributing differently to the overall permeability/ diffusivity of the concrete. Among these, the gel pores, because of their minute size (9 nm in diameter), have basically no effect. The permeability/diffusivity is dependent also on the connectivity of the pores. Since the air voids are usually isolated (except in honeycombs), they also have little effect. Relatively, the capillary pores, which are much larger in size compared to the gel pores and are generally inter-connected in the form of capillaries, have the greatest effect on permeability/diffusivity.

Since the capillary pores are essentially the original space occupied by the free water in the concrete mix remaining unfilled by gel products, the amount of capillary pores in the concrete is dependent mainly on the water/cementitious ratio and the degree of hydration (in turn, degree of hydration is dependent on curing). A sufficiently low water/cementitious ratio together with good curing will produce a concrete with a relatively low permeability/diffusivity for high durability. The addition of supplementary cementitious materials (i.e, pozzolanic materials), such as pulverized fuel ash and condensed silica fume, can also help to reduce the permeability/diffusivity of the concrete.

On the specification of concrete for durability, the British Standard BS8110: Part 1: 1997 refers extensively to another British Standard BS5328. In this regard, it is noteworthy that EN206 has replaced the British Standard BS5328, which has now been withdrawn. However, as EN206 generally deals with concrete mixes based on cylinder strengths - the norm in Europe, the British Standards Institution has produced an accompanying code of practice BS8500, which interprets the recommendations of EN206 in terms of cube strengths. Therefore, where reference is made to EN206, the accompanying British Standard BS8500 should also be referred to.

### 4.2.2 Design for durability

Apart from specifying the right materials to be used and appropriate concrete covers to be provided, the other issues to be considered include:

(a) designing the structure to minimize uptake of water or exposure to moisture;
(b) designing the structure to avoid non-structural cracking;
(c) designing the structure to minimize structural cracking;
(d) adding, where necessary, crack distribution reinforcement to control crack widths;
(e) providing, if necessary, additional protection measures such as cathodic protection, protective coating to the concrete structure and/or protective coating to the steel reinforcing bars etc.

Since many processes of deterioration of a reinforced concrete structure occur only in the presence of water or moisture, it is important to design the structure to avoid ponding and rundown of water. In other words, as far as practicable, the structure should be designed to have good drainage everywhere in the structure including both external and internal areas. In this regard, allowance should be made when necessary to compensate for the effect of long-term deflection of the structure by applying pre-cambering or by increasing the drainage fall. If ponding/rundown of water is somehow unavoidable, such as in bathroom or kitchen areas, considerations should be given to the possibility of providing waterproofing. At locations likely to remain damp for long periods of time, e.g. at confined places where moisture tends to be trapped or near drainage pipes which may have future leakage problems, it may be prudent to provide extra concrete cover to the steel reinforcement.

As mentioned before, non-structural cracks are due to sedimentation, shrinkage or thermal movement of the concrete, which if restrained, would induce tensile stresses large enough to cause cracking. Non-structural cracks can appear during the plastic, curing and long-term stages. Those appearing during the plastic stage include plastic settlement cracks (due to external restraint of the sedimentation movement of the fresh concrete mix by the reinforcing bars and the formwork) and plastic shrinkage cracks (due to self-restraint of the drying shrinkage movement of the top surface of the fresh concrete because of rapid evaporation). Plastic cracking can generally be avoided by improving the mix design of the concrete (basically reducing the water content and increasing the fine powder content) so as to reduce sedimentation and by shielding the top surface of the fresh concrete from direct sunshine and wind so as to prevent rapid evaporation.

Non-structural cracks appearing during the curing stage are mainly early thermal cracks. They are due to internal or external restraints against the thermal movement of the concrete as the temperature of the concrete changes due to the heat generated from the chemical reactions of the cementitious materials. Internal restraint (also called self-restraint) is the major cause of cracking in massive concrete structures while external restraint is the major cause in concrete structures cast against rigid movement restraints. If internal restraint is the major cause, then insulation would help to reduce thermal cracking. However, if external restraint is the major cause, no insulation should be applied as insulation would actually aggravate the problem. Many engineers do not distinguish between the two types of restraints and specify insulation to be applied in all cases. This practice is wrong and is the root cause of many early thermal cracking problems in Hong Kong. In any case, regardless of whether internal and/or external restraints exist, internal cooling by air/water/liquid nitrogen would help to mitigate the early thermal cracking problem, but most contractors do not like this because of the trouble and cost involved, especially if the cooling is not separately priced for as a bill of quantity item by itself. Apart from applying internal cooling, the thermal cracking problem may also be mitigated to some extent by employing a concrete mix (one with a substantial portion of the cement replaced by pulverized fuel ash or ground granulated blastfurnace slag) that would generate less heat during curing. A detailed discussion on whether or not insulation should be applied is available in the following article:
Kwan A.K.H. and Ng I.Y.T., "Avoidance of early thermal cracking in concrete structures: to insulate or not to insulate?", Hong Kong Engineer, February, 2004, pp15-16.

Non-structural cracks appearing during the long-term stage include thermal movement cracks and shrinkage movement cracks. They are due to the thermal/ shrinkage movement of the concrete structure being restrained by rigid walls or supports. Such kind of cracking is a common problem for long podium structures with several tower blocks on top supported by rigid core walls, which form parts of the podium structures and restrain the thermal/shrinkage movement of the podium decks. One way of alleviating the problem is to provide movement joints. If shrinkage movement is the major cause of cracking, then the provision of late-cast strips and the addition of shrinkage reducing agent to the concrete mix should also be considered as possible mitigation measures.

On the other hand, structural cracks are due to tensile stresses developed in the concrete resulting from the applied loads. They are almost unavoidable, unless the structure is to be completely redesigned or prestressed, which is usually not practicable. Nevertheless, the crack widths can be controlled by limiting the tensile stresses developed in the concrete and by proper reinforcement detailing.

Regarding the addition of crack distribution reinforcement for controlling crack widths, it should be noted that it does not stop cracking of the concrete; it only distributes the cracks so that there will be more cracks formed each with a smaller crack width.

For the additional protection measures, which have not been covered in the code, specialist literature should be consulted.

### 4.2.3 Exposure conditions

The exposure conditions are classified in the new code as:

| | |
|---|---|
| Exposure condition 1 – mild | (based on environment) |
| Exposure condition 2 – moderate | (based on environment) |
| Exposure condition 3 – severe | (based on environment) |
| Exposure condition 4 – very severe | (based on environment) |
| Exposure condition 5 – abrasive | (based on mechanical wear and tear) |

The above classification system is based on the guidance given in BS8110 and EN206. However, the principles of classification have been modified to reflect the local conditions (e.g. freeze/thaw exposure and de-icing salts exposure are not of concern in Hong Kong whereas conditions of high humidity and marine environment are). Moreover, the number of exposure conditions based on environment has been reduced for simplification. In BS8110, there are five exposure conditions classified according to the environmental conditions: mild, moderate, severe, very severe and extreme, but in the new code, there are only four exposure conditions classified according to the environmental conditions: mild, moderate, severe and very severe. Therefore, the mild, moderate, severe and very severe exposure conditions in the new code are not quite the same as the corresponding exposure conditions in BS8110. For example, the mild condition (exposure condition 1) in the new code covers slightly more than just the mild condition in BS8110 or the condition XC1 in EN206, as the exposure condition 1 in the new code applies also to concrete surfaces inside buildings protected from the effects of condensation or humidity (a normal condition of Hong Kong buildings), which is similar to the condition XC2 in EN206. Consequently, the durability requirements of the respective exposure conditions in the new code are slightly different from those in BS8110.

### 4.2.4 Cover

The concrete cover to the steel reinforcement (including coupler if used), serves the following functions:

- to protect the steel against corrosion;
- to protect the steel against fire;
- to provide sufficient embedment for safe transmission of bond forces;
- to allow the coarse aggregate particles of the concrete mix to pass through the gap between the mould and the steel reinforcement;
- to allow for unevenness of the surfaces that the concrete will be cast against and future changes in dimensions due to surface treatments such as bush hammering.

Because of the multi-functions of the concrete cover, corrosion protection is only one of the considerations in specifying the required concrete cover.

As in BS8110, the nominal cover concept is used. Nominal cover is the design depth of concrete cover to all steel reinforcement, including links. It is the dimension used in design and indicated on the drawings. The actual cover to all reinforcement should not be less than the nominal cover minus 5 mm. Such an approach is adopted to ensure that during construction, an adequate minimum cover to reinforcement, which is paramount for corrosion protection, can be achieved. The nominal cover specified in the new code is therefore not the minimum cover as stipulated in Building (Construction) Regulations or Code of Practice for Structural Use of Concrete: 1987. There is a 5 mm difference between nominal cover and minimum cover.

The required nominal covers for corrosion protection under different exposure conditions are specified in Table 4.2 of the code. Along with the required nominal cover, the lowest grade of concrete, the maximum free water/cement ratio and the minimum cement content are also given in the table. It should be noted that the nominal cover, lowest grade of concrete, maximum water/cement ratio and minimum cement content together form a "package". They all have to be specified and checked for compliance at the same time.

Taking into account the slight difference in the classification of exposure conditions, the nominal cover, maximum water/cement ratio and minimum cement content etc specified in Table 4.2 of the code are generally in line with the recommendations given in EN206, BS8500 (the British Standard that accompanies EN206 and adapts it for specification in terms of cube strengths) and the report "Specifying concrete to BSEN206-1/BS8500" published by the UK Quarry Products Association.

In Hong Kong, concrete suppliers tend to use higher cement content and lower water/cement ratio than required. Consequently, the concrete strength achieved is usually higher than the specified strength. Given the relatively small difference in cost, such an approach is prudent to ensure the provision of an adequate safety margin to the concrete strength. As a result, a higher concrete strength is usually available that can be beneficial for protecting the steel reinforcement against corrosion.

The addition of supplementary cementitious materials (also called mineral admixtures, such as pulverized fuel ash, ground granulated blastfurnace slag and condensed silica fume etc) to concrete mixes is becoming increasingly popular. Addition of these supplementary cementitious materials can improve the overall performance of the concrete and should be encouraged. Although not explicitly stated, any such supplementary cementitious materials added may be counted towards the cement content for satisfying the maximum water/cement ratio and minimum cement content requirements stipulated in Table 4.2. In actual fact, it has been stated in Clause 4.2.6.3 of the code that when pulverized fuel ash is used, the total content of cement plus pulverized fuel ash should be at least as great as the values given in Tables 4.2 and 4.4 and that in these tables, the word "cement" in "cement content" and "water/cement ratio" means the total content of cement plus pulverized fuel ash, or in other words, the total cementitious materials content. This point should be noted when checking compliance with Tables 4.2 and 4.4.

The addition of plasticizer or superplasticizer to concrete mixes is also becoming quite popular. Addition of these admixtures can improve the workability of the concrete mix and allow the water/cement ratio or cement content of the concrete mix to be reduced for the same workability requirement. The minimum cement content requirement is to ensure proper consolidation of the concrete mix. However, with the addition of plasticizer or superplasticizer, a good workability can be achieved at relatively low cement content. Hence, the minimum cement content should be dependent on whether plasticizer or superplasticizer has been added. Pending more research studies to provide experimental support, the minimum workability of the concrete mix should be specified instead.

Although the exposure condition 4 (very severe environmental condition) in the new code embraces also marine environment, for the specification of marine concrete (i.e. concrete for use in marine environment), the Recommended Specification for Reinforced Concrete in Marine Environment in the Port Works Design Manual: Part 1: 2002 may also be considered. In this recommended specification, the requirements for nominal cover, lowest grade of concrete, maximum water to total cementitious materials ratio and cementitious materials content are 75 mm, C45, 0.38 and 380-450 kg/m$^3$ respectively. Except for the required lowest grade of concrete, these requirements are similar to or slightly more stringent than the respective requirements in the new code. In fact, based on the author's own experience, the restriction of the water to total cementitious materials ratio at not higher than 0.38 would lead to a concrete grade of at least C50, which is actually in line with the new code.

### 4.2.5 Concrete materials and mixes

Under certain specific conditions, the code allows the lowest concrete grade to be reduced by not more than 5 provided the maximum water/cement ratio and minimum cement content requirements are proven to have been met and allows the minimum cement content to be reduced by not more than 10% provided the corresponding water/cement ratio is reduced by not less than the percentage reduction in the cement content. Tighter quality control and a more systematic checking regime are required when these reductions are to be applied. These allowable reductions are derived mainly from BS8110. In Hong Kong, there should be no particular difficulties in achieving the required strength grade and minimum cement content. Hence, in practice, there is seldom the necessity to make use of these allowable reductions.

When the maximum aggregate size is smaller than or larger than 20 mm, the code requires the minimum cement content to be adjusted upwards or downwards, respectively. As mentioned before, this is to ensure proper consolidation of the concrete mix. However, since the workability of the concrete mix can always be improved by adding plasticizer/superplasticizer, there is no longer the necessity to impose any minimum cement content requirement. Pending more research studies to provide experimental support, the minimum workability of the concrete mix should be specified instead. Such performance specification would allow more flexibility on the side of the concrete producer and easier control on the side of the engineer responsible for site supervision.

In the code, the usual range of pulverized fuel ash (PFA) is 25% - 35% by mass of the total cementitious materials content, although the Practice Note PNAP APP-33 limits the amount of PFA to 25%. Both the Housing Department and MTRC specifications have been allowing the addition of PFA up to 35% and their correspondings performance are satisfactory. It is considered appropriate to increase the permissible level of PFA to 35%

On one hand, the addition of PFA can reduce the permeability/diffusivity of the concrete by converting the soluble lime (a by-product of the chemical reaction between cement and water) in the concrete to further gel products. On the other hand, it may also reduce the alkalinity of the concrete, which is needed for maintaining the passivation protection to the steel reinforcement. Overall, the durability performance of concrete made with PFA is about the same as that of concrete made without PFA, provided the concrete with PFA is of the same strength grade as the concrete without PFA. However, concrete with PFA added generally develops strength at a slower rate and requires a longer period of curing. More attention to the curing and removal of formwork is required with concrete mixes containing PFA.

The usual range of granulated blastfurnace slag (GGBS) is 35% - 75% by mass of the total cementitious materials content. The ingredients of GGBS are similar to Portland cement which consist of lime, silica, alumina and magnesia, thus Portland cement could be replaced by GGBS in higher percentage. Meanwhile, using of concrete containing GGBS could improve corrosion and sulphate resistance in order to enhance the durability of a concrete structures.

Whereas, GGBS is formed by glassy materials. The strength development of concrete with GGBS would be slower than that of concrete without GGBS. Cements with a high content of GGBS can be used as low heat cements in order to control the temperature inrease arising from the early development of heat of hydration.

### 4.2.6 Mix proportions

As a basic principle, the total cementitious content is limited to not more than 550 kg/m$^3$ in order to maintain a high dimensional stability (i.e. small changes in dimension due to early temperature rise during curing, drying shrinkage at the long-term stage and creep under sustained loading).

For normal concrete, i.e. $f_{cu} \leq 60\,\mathrm{N/mm^2}$, which is relatively easy to produce without the use of a high total cementitious content, the above maximum limit of 550 kg/m$^3$ should be strictly applied and should not be exceeded unless there is a proven need and special considerations have been given to the risks of early thermal and drying shrinkage cracking and to the possible adverse effects of creep.

For high-strength concrete, i.e. $f_{cu} > 60\,\mathrm{N/mm^2}$, which is generally more difficult to produce and requires the use of a higher total cementitious content, the imposition of the above maximum limit of 550 kg/m$^3$ might be too restrictive. Nevertheless, if the total cementitious content has to exceed 550 kg/m$^3$, special considerations should be given to the risks of early thermal and drying shrinkage cracking and to the possible adverse effects of creep. Moreover, under normal circumstances, the cement content (note: not the total cementitious content) should be limited to not more than 450 kg/m$^3$.

Regardless of whether the total cementitious content of the concrete mix is on the high side or not, it is recommended that for massive concrete structures, temperature rise evaluation test should be carried out to determine the temperature rise of the concrete, for long concrete structures, drying shrinkage test should be carried out to determine the ultimate shrinkage strain of the concrete, and for concrete structures likely to be affected by creep effects, creep test should be carried out to determine the creep coefficient of the concrete.

### 4.2.7 Mix constituents

Since the presence of chloride in concrete may cause de-passivation of the steel reinforcement and may affect the sulphate resistance of the concrete, it is necessary to control the chloride content in the concrete mix. No admixture containing any chloride should be used. The total chloride content in the concrete mix, expressed as a percentage of chloride ion by mass of the total cementitious materials content (including both cement and PFA), should not exceed 0.1, 0.2 and 0.35 for prestressed concrete or steam-cured concrete, concrete made with sulphate resisting cement, and concrete containing steel or other metals, respectively. These values are slightly more stringent than those permitted in BS8110: Part 1 (same as those in BS5328) and EN206 and are specifically developed to cater for the aggressive environment in Hong Kong.

Though not common, deleterious chemical reactions between the aggregate particles and the surrounding hardened cement paste, leading to disruption and cracking of the concrete, have been observed in Hong Kong. The most common chemical reaction is that between the active silica constituents of the aggregate and the alkalis in the cement. Such reaction is called *alkali-silica reaction*. Another type of deleterious aggregate reaction is that between some dolomite limestone aggregates and the alkalis in the cement. This is called *alkali-carbonate reaction*. Any aggregate that may be susceptible to alkali-silica reaction or alkali-carbonate reaction (collectively known as alkali-aggregate reaction) should not be used. The risk of alkali-aggregate reaction may be reduced by:

- limiting the amount of alkalis in the concrete mix, and
- adding pozzolanic materials such as pulverized fuel ash and condensed silica fume, which react with the lime in the concrete and thereby consume part of the alkalis.

The guidelines given in the code are based on the Practice Note PNAP APP-74.

### 4.3 Requirements for fire resistance

The Code of Practice for Fire Resisting Construction is referred to regarding the nominal cover and minimum dimensions of members required for fire resistance.

Fire resistance could be defined as the ability of an element of building construction to fulfil its designed function during fire exposure. There are three methods to assess the fire resistance of concrete which are (i) fire testing, (ii) prescriptive method and (iii) performance-based method. The Code does not include the fire-resistance design for concrete structure and only provides the fire limit states and different load factors for the dead load and imposed load. However, the Code does not tally with the Eurocode 2; the reduction factors of loading under fire exposure and design procedures not included. The design methods from Eurocode could be a reference.

The Code are conceptually similar to structural design for normal temperature conditions. The main differences of fire limit state and normal temperature designs are (i) the applied loads, (ii) strength of material at elevated temperatures, (iii) reduction of cross sectional area by spalling, (iv) safety factor because of low likelihood of the event and (v) consideration of different failure mechanisms.

Because of its low permeability to water and vapour, a high-strength concrete subjected to prolonged heating during fire tends to have a high vapour pressure developed within its pores. Hence, high-strength concrete is more susceptible to spalling failure than normal-strength concrete during fire attack.

The code requires that the reduction of strength at elevated temperature should be properly considered. There are four methods for preventing spalling which tally with the requirement of Eurocode 2 Part 1-2.

In the case of concrete compressive strength greater than 60MPa, there will be a reduction of strength at elevated temperatures account for risk of spalling. The protection measures for spalling of high-strength concrete are also provided. It provides four methods to reduce the risk of concrete spalling which are:

1. A reinforcement mesh with a nominal cover of 15mm. This mesh shall have wires with a diameter ≥ 2mm with a pitch ≤ 50 x 50mm. The nominal cover to the main reinforcement shall be ≥ 40mm;
2. Include in the concrete mix not less than 1.5kg/$m^3$ of monofilament propylene fibres. The fibres shall be 6 - 12mm long and 18 - 32$\mu$m in diameter, and shall have a melting point less than 180°C;
3. Protective layers for which it is demonstrated by local experience or fire testing that no spalling of concrete occurs under fire exposure;
4. A design concrete mix for which it has been demonstrated by local experience or fire testing that no spalling of concrete occurs under fire exposure.

For high strength concrete exceeding C80, at least one fire test should be carried out to demonstrate that the main reinforcing bars of a structural member shall not be exposed during the design fire resistance rating.

# 5 STRUCTURAL ANALYSIS

## 5.1 General provisions

### 5.1.1 General provisions

For *analysis of structure*, the properties of materials may be assumed to be those associated with their *characteristic strengths* (note: not the design strengths), irrespective of which limit state is being considered. The reason behind is that the characteristic strengths are more realistic estimates of the actual strengths of the materials. Although 5% of the materials may have their strengths lower than their characteristic strengths, about 95% of the materials should have their strengths higher than their characteristic strengths. The purpose of analysis of structure is to determine the internal force distributions, which are dependent on the actual properties of the materials. Use of conservative estimates (or factored down estimates) of the actual strengths of the materials in analysis of structure does not necessarily yield an internal force distribution that is on the safe side for design. For instance, the use of a relatively small stiffness of a beam calculated from conservative estimates of the strengths of the materials in analysis of structure would tend to underestimate the internal forces induced in the beam thereby leading to unsafe design of the beam.

For *analysis of sections*, the properties of materials should be assumed to be those associated with their *design strengths* (i.e. the characteristic strengths divided by respective material safety factors) appropriate to the limit state being considered. The reason is that although the average strengths of the materials should be close to, if not higher than, their characteristic strengths, locally, at a given section, there is a 5% probability that the strengths of the materials are lower than their respective characteristic strengths. Hence, to be on the safe side, the design strengths should be used in analysis of sections.

The structural model to be used for analysis should truly represent the actual behaviour of the structure as far as practicable. If there are uncertainties in any of the structural parameters, such as lack of fitness, extent of cracking, and soil and water pressures etc., the possible ranges of the structural parameters should be estimated and the analysis of structure carried out repeatedly using different combinations of possible values of the structural parameters. This is to ensure that the worst scenario could be catered for in the design.

In stage-by-stage construction, the construction sequence and schedule could have significant effects on the behaviour of the structure. These effects could arise from the deformation/movement of the partially completed structure due to permanent and/or temporary loading, prestressing, thermal expansion/contraction, drying shrinkage or creep. Any such effects should be fully allowed for by simulating the construction sequence and schedule in the analysis of structure.

### 5.1.2 Methods of analysis

As stated in the code, the objective of analysis of structure is to determine the distribution of internal forces that are *in equilibrium* with the *design loads* for the required load combinations. The following two points should be noted:

(a) the internal forces must be in equilibrium with the applied loads; and
(b) the applied loads are to be taken as the design loads (i.e., the characteristic loads multiplied by the load safety factors).

Whatever method used and no matter how rigorous or sophisticated the method is claimed to be, the equilibrium condition everywhere in the structure, i.e. within every member and at every joint, must be satisfied. There is no compromise on this fundamental condition. This is particularly important when computer methods are used for the analysis. For example, when applying the finite element method to the analysis of structures, it should be borne in mind that the accuracy of the numerical results is dependent on the fineness of the finite element mesh employed to model the structure. The finite element method is an approximate method and the numerical results converge to the correct results only when the mesh size is reduced to nearly zero. If a coarse mesh is employed to model the structure, the computer results may not have sufficient accuracy (however, the author has seen some engineers using very coarse meshes for finite element analysis without ever checking the convergence of the numerical results). Moreover, the stresses evaluated from the shape functions of the elements are not necessarily in equilibrium with the applied loads. Nevertheless, if the mesh used is fine enough to obtain convergent results, the stresses evaluated from the shape functions are nearly in equilibrium with the applied loads. Hence, it should be a good practice to ascertain the correctness and accuracy of the finite element analysis results by manually checking whether the global equilibrium of the structure and the local equilibrium of the key elements in the structure are satisfied. Such practice can also help to identify mistakes in data entry.

In the analysis of structure, although the properties of materials (for the purpose of structural modelling) are to be taken as those associated with their *characteristic strengths*, the applied loads are to be taken as the *design loads*. They are not incompatible because the characteristic strengths are actually better estimates of the actual strengths than the design strengths. The design strengths are only used for analysis of sections to cater for the possibility of having locally substandard materials in the structure.

In ULS design, redistribution of forces and moments within the structure for the purpose of exploiting the plastic capacity of the structure (when a section of the structure yields, the structure would not normally fail immediately but the additional forces acting on the yielded section would be redistributed to the other unyielded sections until a collapse mechanism is formed) is allowed provided it can be shown that all the yielded sections have sufficient ductility to withstand the plastic deformation so caused until the structure collapses. For checking whether such ductility demand can be met by all the yielded sections, a rigorous limited ductility elasto-plastic analysis is needed. Nevertheless, the redistribution of forces and moments explicitly permitted in Chapter 5 and provision of ductility detailing in Section 9.9 of the code may be applied without resorting to such rigorous limited ductility analysis.

In SLS design, a linear elastic analysis would normally suffice. Three methods of evaluating the section stiffness of the members are given in the code. They are all regarded as acceptable but a consistent method should be applied to all members of the structure. This is because if the method used to evaluate the section stiffness tends to overestimate the section stiffness, it would overestimate the section stiffness of all members so that the relative stiffness of the members would remain more or less the same. In the analysis of structure for internal force distribution, it is the relative stiffness of the members that matters, not the absolute values of the section stiffness of the members. However, if the analysis of structure is to determine the maximum deflection, then there will be a small difference in the deflection results when different methods are used to evaluate the section stiffness.

### 5.1.3 Load cases and combinations

The load combinations to be considered in the design of beams and slabs and in the design of columns and walls have been given in the code for the relatively simple cases only.

For more complicated cases, such as beams subjected to moment, shear and torsion, columns subjected to axial load and biaxial bending, core walls subjected to axial load, biaxial bending and torsion, and frames with bracing members etc., no general guidance can be given. The design engineer will have to exercise his/her own judgement on the load combinations to be considered in the design so that no critical loading case is omitted.

It should be noted that the load combinations for the design of beams and slabs specified in the code are not the same as those given in BS8110 or Eurocode 2. In BS8110, the load combinations to be considered are:

(a) all spans loaded with the maximum design load; and
(b) alternate spans loaded with the maximum design load and all other spans loaded with the minimum design load.

In Eurocode 2, the load combinations to be considered are:

(a) alternate spans loaded with the maximum design load and all other spans loaded with the minimum design load; and
(b) any two adjacent spans loaded with the maximum design load and all other spans loaded with the minimum design load.

In the code, the load combinations to be considered are:

(a) all spans loaded with the maximum design load;
(b) alternate spans loaded with the maximum design load and all other spans loaded with the minimum design load; and
(c) any two adjacent spans loaded with the maximum design load and all other spans loaded with the minimum design load.

Particular attention should also be paid to the load combinations for the design of columns and walls specified in the code. In both BS8110 and Eurocode 2, there is no recommendation on the load combinations to be considered, but in the code, the following load combinations are recommended:

(a) maximum axial load combined with coexistent bending moment;
(b) minimum axial load combined with coexistent bending moment;
(c) maximum bending moment combined with coexistent axial load; and
(d) any other coexistent combinations of axial load and bending moment which will be more critical than the above cases.

### 5.1.4 Imperfections and second order effects

The second order effects are not expected to be significant at the SLS. Hence, these only need to be considered at the ULS. The requirement and method of incorporating the second order effects in the analysis of structure are stipulated in Section 5.3 of the code and will be explained later in due course.

## 5.2 Analysis of structure

### 5.2.1 Idealisation of the structure

Rules for the classification of structural elements into the following types have been provided:

- beams/deep beams (based on CIRIA Guide No. 2);
- slabs/one way slabs (same as in Eurocode 2);
- ribbed/waffle slabs not treated as discrete elements (same as in Eurocode 2);
- columns/walls (same as in Eurocode 2).

However, there are no rules for the classification of structural elements into core walls, coupling beams, arches and shells etc.

Apart from the above, the provisions given in Clause 5.2.1 of the code are the same as those given in Clause 5.3.1 of Eurocode 2.

For T and L beams, a formula for evaluating the effective flange width $b_{\text{eff}}$, over which uniform conditions of stress can be assumed, has been provided as follows:

$$b_{\text{eff}} = \sum_{i=1}^{2} b_{\text{eff},i} + b_w \qquad \text{Equation 5.1}$$

where $b_{\text{eff},i} = 0.2b_i + 0.1l_p \leq 0.2l_p$ and $b_{\text{eff},i} \leq b_i$. The effective flange width is to allow for shear lag in the flange, which causes decreasing bending stress away from the web. Note that the effective flange width $b_{\text{eff}}$ is dependent on the type of loading, span length and support conditions because the distance between points of zero moment $l_p$ is dependent on these parameters.

The Code provides additional provision in the case of $b_{\text{eff},i} > 0.1l_{pi}$, the shear stress between the web flange should be checked and provided with transverse reinforcement. This allows the Engineer to use either the shorter BS 8110 flange width or the longer EC 2 flange width that requires checking of shear.

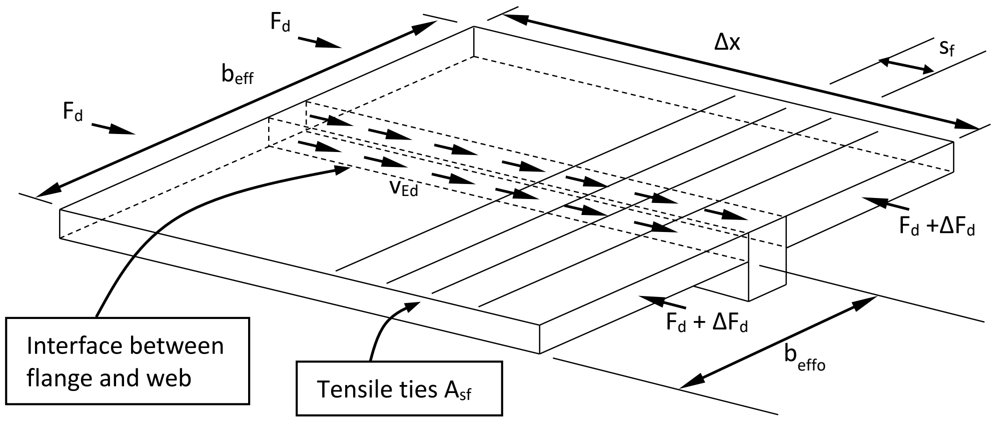



Figure 5.1 Shear between flange and web

Longitudinal complementary shear stresses occur in a flanged section along the interface between the web and flange as shown in Figure 5.1. Transverse reinforcement shall be provided over the effective width of the flange beam.

The longitudinal shear stresses are at a maximum in the region of maximum change in bending stresses which occurs at the steepest part of the bending moment diagram. The change in longitudinal force $\Delta F_d$ in the flange outstand a section is obtained from

$$\Delta F_d = \frac{\Delta M}{\left(d - h_f/2\right)} \times \frac{b_{effo}}{b_{eff}}$$

Where $b_{eff}$ = effective breath of flange

$b_{effo}$ = the breath of the outstand of the flange = $(b_{eff} - b_w)/2$ (Assume that $b_{effo}$ are the same on both sides)

$b_w$ = breath of web

$h_f$ = the thickness of the flange

$\Delta M$ = Change of moment over the distance $\Delta x$

The longitudinal shear stress, $v_{Ed}$, at the vertical section between the outstand of the flange and web is due to the change in the longitudinal force $\Delta F_d$, which occurs over the distance $\Delta x$.

$$v_{Ed} = \frac{\Delta F_d}{\left(h_f \times \Delta x\right)}$$

The maximum value allowed for $\Delta x$ is half the distance between the section with zero moment and that where maximum moment occurs. The required transverse reinforcement per unit length, $A_{sf}/s_f$, may be calculated from the equation:

$$\frac{A_{sf}}{s_f} = \frac{v_{Ed} \times h_f}{0.87 f_y}$$

A worked example is presented in section 6.1.2.

A formula for evaluating the effective span *l* of a beam or slab has also been given in the code as follows:

$$l = l_n + a_1 + a_2 \qquad \text{Equation 5.4}$$

where $l_n$ is the clear distance between the faces of the supports, and $a_1$ and $a_1$ are each equal to min. $\left(h/_2 , {}^{S_w}/_2\right)$ at the respective end of the span. However, it should be noted that these effective spans are provided mainly for member analysis (i.e. for analysing the bending moment and shear force along the beam or slab when subjected to vertical load only). For frame analysis, the code states that these assumed effective spans should be used only where appropriate. It is suggested herein that for frame analysis, the effective span of a beam or slab spanning over two adjacent supports should be taken as the distance between the centrelines of the two adjacent supports.

For any support, unless rotational restraint has been properly provided (and structurally designed and detailed as such), the support should be considered as not providing any rotational restraint and treated as a hinge support.

Where a beam or slab is monolithic with its supports to provide rotational restraints, the critical design moment at the support may be taken as that at the face of a rectangular support, or at $0.2\phi$ inside the face of a circular support of diameter $\phi$, but should not be taken as less than 0.65 of the full fixed end moment, as shown in Figure 5.2.

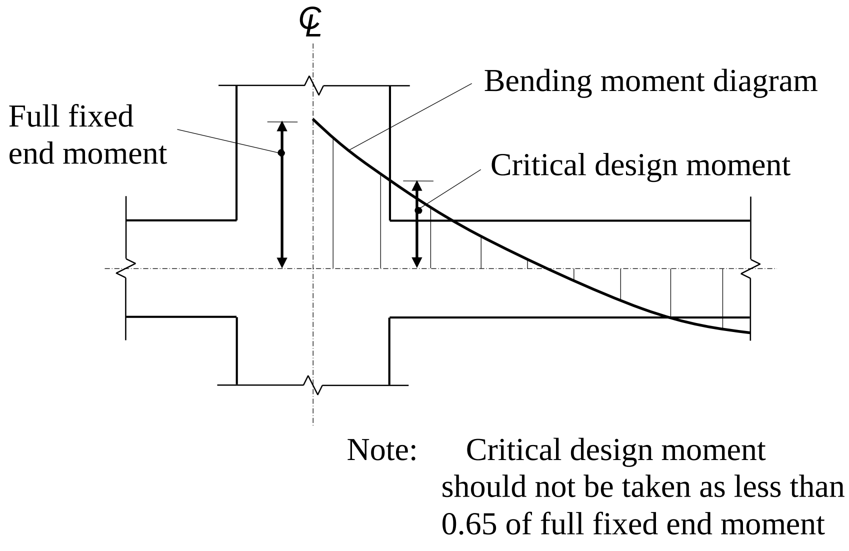



Figure 5.2: Critical design moment at a monolithic support

Where a beam or slab is spanning continuously over a support, which may be considered as not providing any rotational restraint, the design support moment, calculated on the basis of a span equal to the centre-to-centre distance between supports, may be reduced by an amount $\Delta M_{\mathrm{Ed}}$ as follows:

$$\Delta M_{\mathrm{Ed}} = {F_{\mathrm{Ed,sup}} S_w}/{8} \qquad \text{Equation 5.5}$$

in which $F_{\mathrm{Ed,sup}}$ is the design support reaction. The value of $\Delta M_{\mathrm{Ed}}$ is estimated by assuming that $F_{\mathrm{Ed,sup}}$ is uniformly distributed across the width $S_w$ of the support, as illustrated in Figure 5.3.

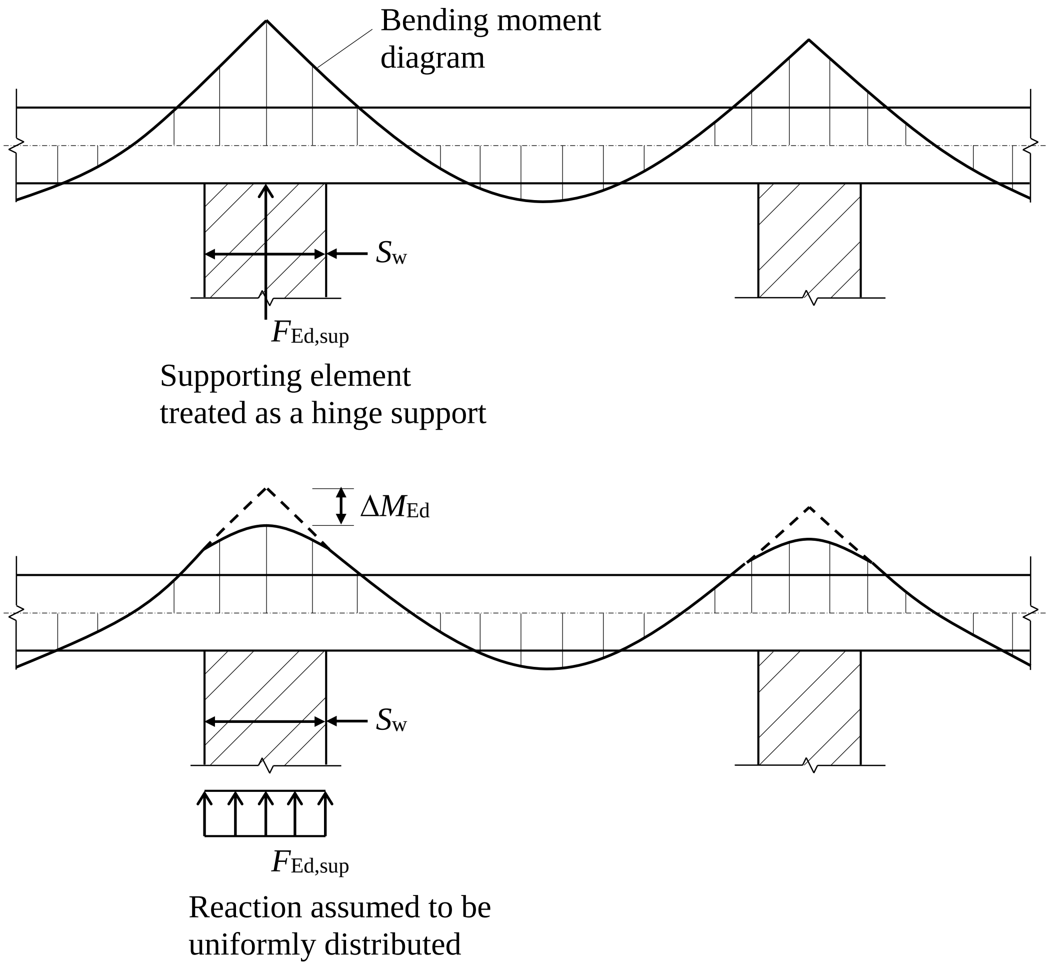



Figure 5.3 Reduction of design support moment

### 5.2.2 Analysis of sections for ultimate limit states

For analysis of sections at ULS, the "plane sections remain plane" assumption may be adopted. Based on this assumption, the bending strain (of both the steel and concrete) within a section varies linearly across the section (note however that the stress-strain relationships are not linear). Basically, the bending strain of a longitudinal fibre (whether steel or concrete) is directly proportional to the distance of the fibre from the neutral axis.

The strength of a section under both short- and long-term loading may be assessed using the stress-strain curves derived from the design strengths of the materials as given in Sections 3.1, 3.2 and 3.3 of the code.

### 5.2.3 Analysis of sections for serviceability limit states

For analysis of sections at SLS, the “plane sections remain plane” assumption may be adopted. In addition, it may be assumed that both the steel and concrete remain elastic so that their stress-strain relationships are linear. Since the bending strain of a longitudinal fibre is directly proportional to the distance of the fibre from the neutral axis and the stress-strain relationships are linear, the bending stress within a section should vary linearly across the section and should be directly proportional to the distance from the neutral axis.

### 5.2.4 Simplifications

Monolithic frames may be simplified into sub-frames, continuous beams or sway-frames, as outlined in Clauses 5.2.5 and 5.2.6 of the code and explained below.

### 5.2.5 Monolithic frames not providing lateral restraint

For a monolithic frame not providing lateral restraint, only the vertical loads need to be considered. It may be analysed as a series of sub-frames, each for the purpose of analysing the beams at one level only, so that the analysis can be carried out on a level-by-level basis. Each sub-frame consists of the beams at the level being considered together with the columns above and below, if any, as shown in Figure 5.3. Each end of the columns remote from the beams may generally be assumed to be fixed unless a pinned end is clearly more reasonable.

As an alternative, a monolithic frame not providing lateral restraint may also be analysed as a series of simplified sub-frames, each for the purpose of analysing an individual beam only, so that the analysis can be carried out on a beam-by-beam basis. Each sub-frame consists of the beam being considered, the columns attached to the ends of the beam and the other beams on either side, if any, as shown in Figure 5.5. Each end of the columns and beams remote from the beam being considered may generally be assumed to be fixed unless a pinned end is clearly more reasonable. The stiffness of the other beams on either side of the beam being considered should be taken as half their actual values if they are taken to be fixed at their outer ends.

As another alternative, a monolithic frame not providing lateral restraint may also be analysed as a series of continuous beams, each for the purpose of analysing the beams at one level only, so that the analysis can be carried out on a level-by-level basis. Each continuous beam consists of the beams at the level being considered spanning over supports not providing any rotational restraint, i.e. hinge supports, as shown in Figure 5.6. Where the beams at one level have been analysed as a continuous beam, the moments in the columns attached to the beams may be calculated by applying simple moment distribution procedures at the beam-column joints, on the assumption that the ends of columns and beams remote from the joint under consideration are fixed and that the beams possess half their actual stiffness. It should be noted that the critical load combinations for the design of columns are in general different from those for the design of beams.

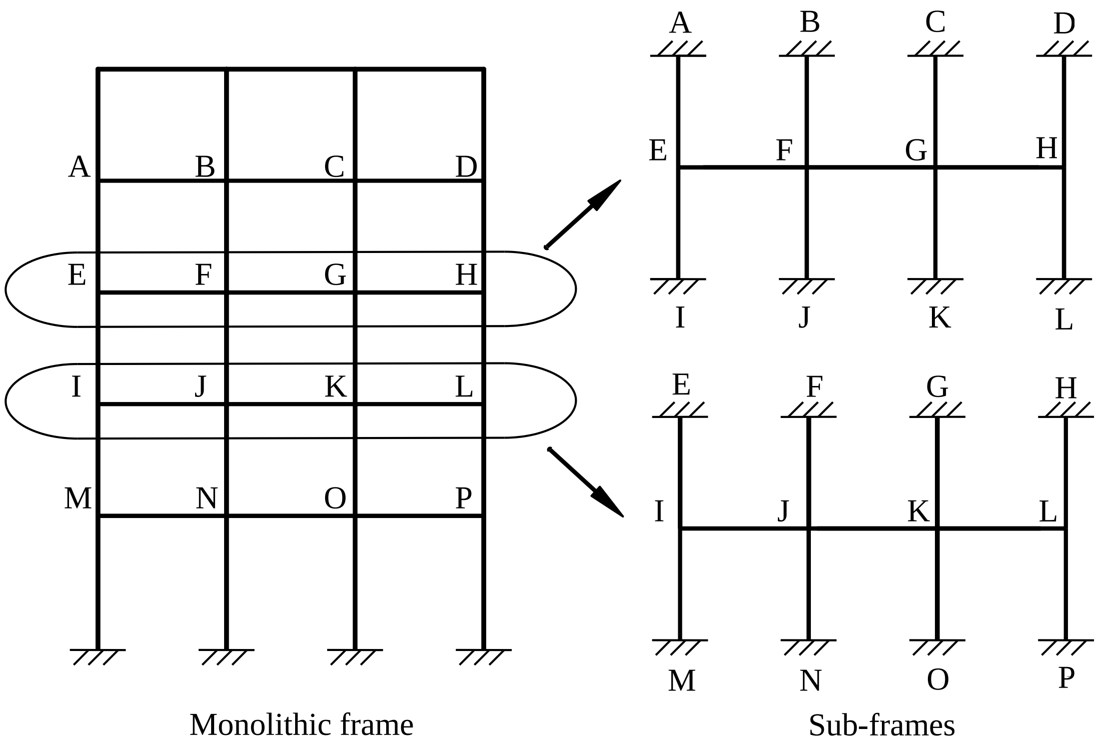



Figure 5.4 Simplification into sub-frames for analysis on level-by-level basis

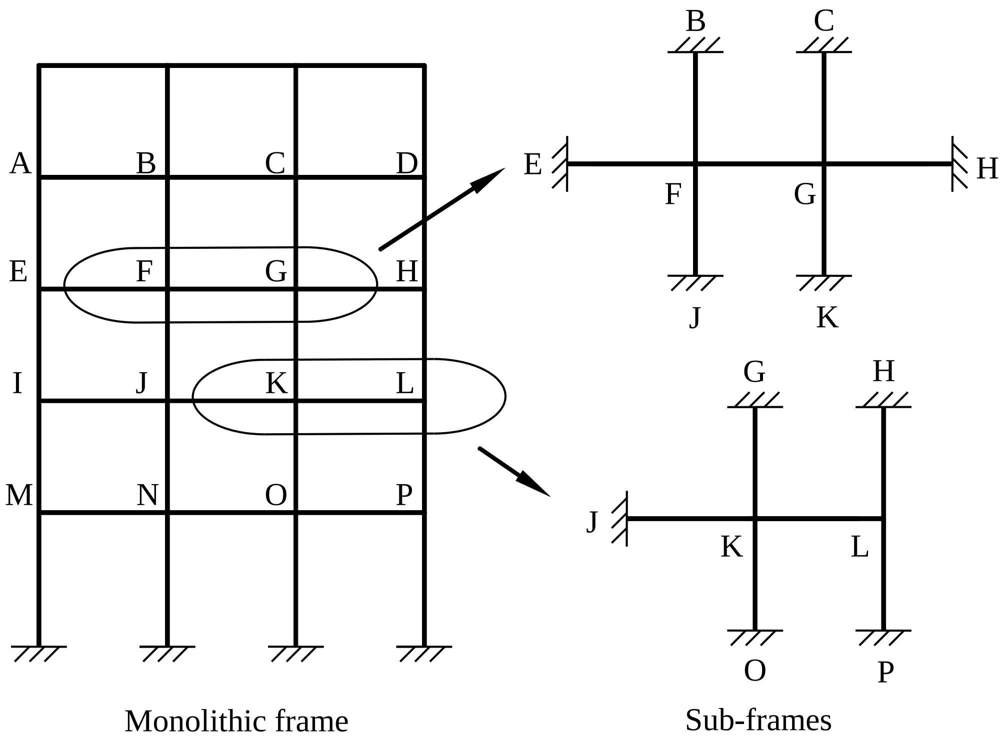



Figure 5.5 Simplification into sub-frames for analysis on beam-by-beam basis

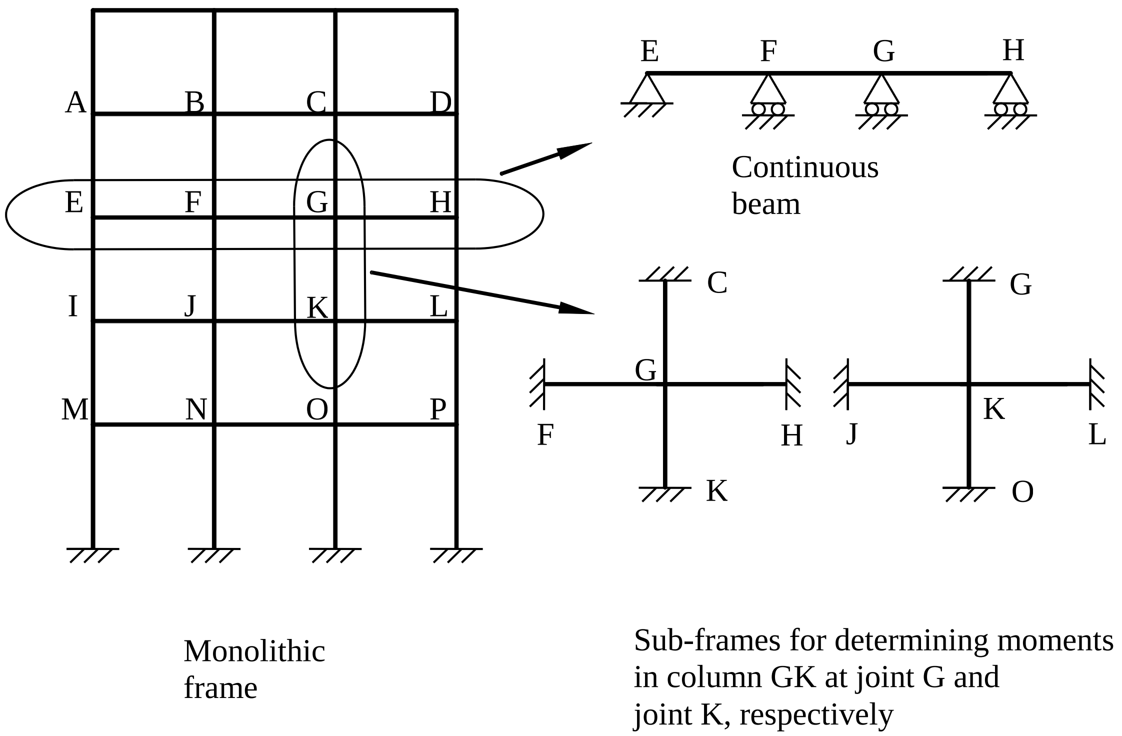



Figure 5.6 Simplification into continuous beams and sub-frames

5.2.6 Frames providing lateral stability

For a monolithic frame providing lateral stability to the structure as a whole (or in other words, helping to resist the lateral loads), the horizontal loads acting on the frame and the resulting sway need also to be considered. In addition, if the columns of the frame are slender, the additional moments arising from the $P$-$\Delta$ effect should be imposed.

The analysis of the frame may be simplified by considering the effects of the vertical loads and the effects of the horizontal loads separately, and then superimposing the effects of the vertical loads and the effects of the horizontal loads together to obtain the individual member forces. For the analysis of the frame subjected to vertical loads only, the same procedures as for a monolithic frame not providing lateral restraint may be applied (e.g. simplifying the frame into sub-frames or continuous beams). For the analysis of the frame subjected to horizontal loads only, the frame may be simplified into a sway-frame by incorporating hinges at the points of contraflexure of all the beam and column members (the point of contraflexure of a beam or column may be assumed to be located at the centre of the member if the member is rigidly connected at both ends to the other parts of the structure or at the lower end of the member if the member is a ground floor column pinned at the ground level), as shown in Figure 5.7.

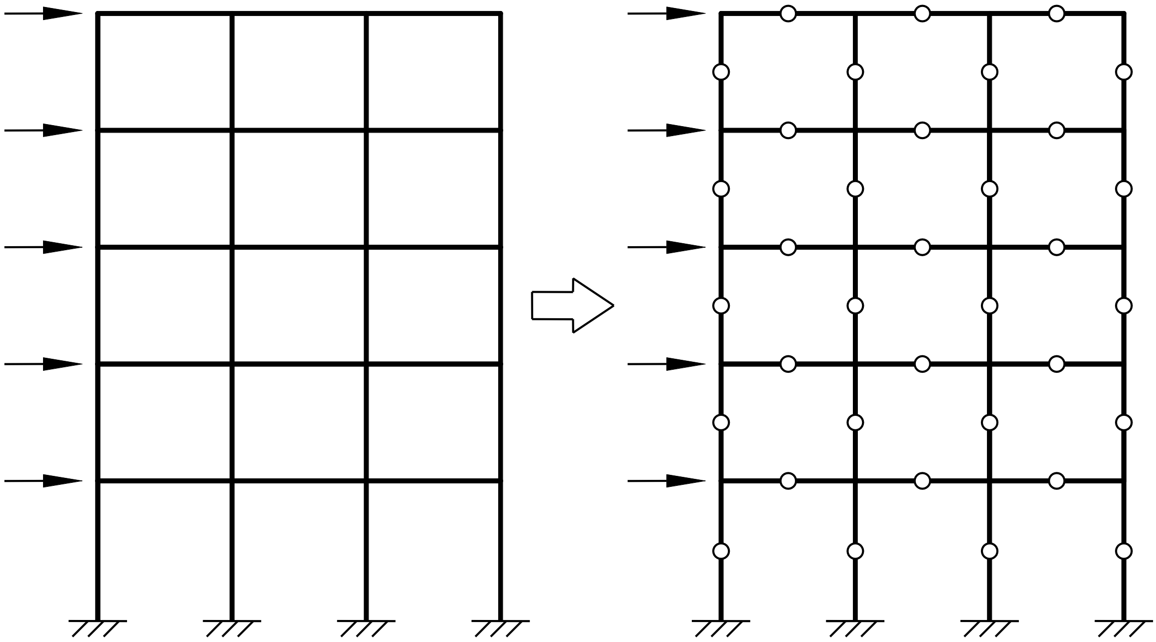



Figure 5.7 Simplification into a sway-frame

In a tall building, one or more shear/core walls should have been incorporated to help resist the lateral loads and to provide partitions and enclosures for the utilities, lifts and staircases etc. With the co-existence of shear/core walls and frames, complex wall-frame interaction would occur and it is not easy to directly determine the amount of horizontal loads that could act on the frame part of the building. Wall-frame interaction occurs mainly because the deflection mode of a wall and the deflection mode of a frame are very different. For example, when a uniform wall is subjected to a horizontal point load at top, its deflection would increase with height as a cubic polynomial function, but when a uniform frame is subjected to a horizontal point load at top, its deflection would increase with height as a linear function. As the deflections of the wall and the frame parts of the building are constrained to be equal by the high in-plane rigidity of the floor slabs, there would be shear transfer between the wall and the frame parts of the building. Roughly speaking, near the top of the building, the storey shear would be transferred mostly to the frame part so that the wall part would be subjected to little storey shear, whereas toward the bottom, the storey shear would be transferred mostly to the wall part so that the frame part would be subjected to little storey shear. For more detailed explanations and in-depth analysis, the following book is recommended (the whole of Chapter 11 of the book is on this topic):
Stafford Smith B. and Coull A., Tall Building Structures: Analysis and Design, John Wiley & Sons, Inc., 1991, 537pp.

Because of wall-frame interaction, the horizontal loads are not distributed to the wall and frame parts of the building in direct proportion to their relative lateral stiffness. There are, in fact, no simple methods for analysing the wall-frame interaction and the distribution of horizontal loads to the various parts of the building structure. The only realistic way of analysing the lateral behaviour of a composite wall-frame structure is by means of computer analysis using either the frame analogy or the finite element method.

Hence, although the code allows the analysis of a frame subjected to horizontal loads only to be carried out by simplifying the frame into a sway-frame, the author does not recommend this overly simplified method to be applied whenever there is any wall in the building. Nevertheless, the practice of considering the effects of the vertical loads and the effects of the horizontal loads separately, and then adding the effects of the vertical loads and the effects of the horizontal loads together to obtain the individual member forces is still applicable, despite the complexity due to wall-frame interaction.

### 5.2.7 Slabs

Provisions for the analysis of slabs are given in Chapter 6 of the code and will be explained later in due course.

### 5.2.8 Corbels and nibs

Provisions for the analysis of corbels and nibs are given in Chapter 6 of the code and will be explained later in due course.

### 5.2.9 Redistribution of moments

Reinforced concrete behaves in a manner intermediate between the elastic-plastic behaviour of the reinforcement and the behaviour of the concrete which, in normal circumstances, is capable of very little plastic deformation. The exact behaviour depends on the relative quantities of the two materials and their properties; however, it may be considered to be roughly elastic until the steel yields and then roughly plastic until the concrete fails in compression. The concrete failure limits the amount of this plastic behaviour or, more specifically, it limits the amount of rotation which a plastic hinge can undergo.

The provisions in the code for redistribution of moments are similar to those given in BS8110: Part 1: 1997, except that an upper limit to the concrete grade beyond which no redistribution of moments is allowed has been set and that the requirement of limiting the neutral axis depth of the member to a smaller value when a higher strength concrete is used has been incorporated.

For redistribution of moments to be applied to an internal moment distribution obtained by elastic analysis, the following conditions must be satisfied:
Condition 1. Equilibrium between internal and external forces is maintained.

Condition 2. Where the design ultimate resistance moment has been reduced, the section subjected to largest moment meets the following:

$$x/d \le (\beta_b - 0.4) \text{ for } f_{cu} \le 45\,\mathrm{N/mm^2} \qquad \text{Equation 6.4}$$

$$x/d \le (\beta_b - 0.5) \text{ for } 45\,\mathrm{N/mm^2} < f_{cu} \le 70\,\mathrm{N/mm^2}$$

Equation 6.5

in which $\beta_b = \frac{\text{moment after redistribution}}{\text{moement before redistribution}}$ Equation 6.6

Condition 3. At any section, the design ultimate resistance moment is at least 70% of maximum moment before redistribution.

Redistribution will also affect the shears. Depending on how the redistribution is done, these may be either decreased or increased. It is considered that it is prudent to take the worst of the redistributed and unredistributed shears as the design values. A reason for this is that, if the redistribution reduces the shears and, possibly because the strength of the reinforcement was above its design value, the redistribution did not take place then design for the reduced shears could lead to the structure having a reduced safety factor.

In addition to the above, for members in a frame providing lateral stability, the redistribution of moments is limited to 10% (i.e. $\beta_b \ge 90\%$), and the design ultimate resistance moment at any section should be at least 90% of the maximum moment of the section before redistribution. The redistribution of moments is limited to 10% because the formation of plastic hinges at the onset of redistribution may induce a premature failure due to frame instability.

## 5.3 Second order effects with axial loads

Second order effects should be taken into account wherever the lateral deflection of the structure could affect the load distribution and equilibrium of any part of the structure. They are more likely to occur in slender elements subjected to axial loads such as columns, walls and piles subjected to vertical loads, arches subjected to both axial compression and bending moment, and plates (plates include slabs) and shells subjected to both in-plane compression and out-of-plane bending.

To take into account the second order effects, the deformed state of the structure should be used for the analysis of structure. Such analysis with the deformation of the structure taken into account may be carried out in an iterative manner. At the beginning, the deformation of the structure may be neglected and the analysis of structure carried out with all the likely imperfections (such as initial curvature and eccentricity) incorporated to determine an initial estimate of the deformation of the structure. Then, using the initial estimate of the deformation of the structure so obtained, the analysis of structure is carried out again for a better estimate of the deformation of the structure. If the better estimate of the deformation of the structure turns out to be significantly larger than the previous estimate, the analysis of structure is repeated each time with the deformation of structure updated until the deformation results converge to more or less constant values or with very little differences between successive values.

It should be noted that after incorporating the second order effects, columns originally designed for axial compression and uniaxial bending only might become subjected to axial compression and biaxial bending and walls originally designed for in-plane loads only might become subjected also to out-of-plane bending.

## 5.4 Shear walls

Shear walls are defined as plain or reinforced concrete walls contributing to the lateral stability of the structure. Care should be taken to differentiate shear walls from infill panels in frames. Both shear walls and infill panels can be plain or reinforced and thus they cannot be differentiated by just looking at whether the wall panels are plain or reinforced. Infill panels are those wall panels in frames, which serve as removable partitions and thus cannot be treated as contributing to the lateral stability of the structure. Although, theoretically, infill panels can be structurally connected to the frames to act as structural elements contributing to lateral stability, since tenants in Hong Kong like to repartition the floor areas by knocking down the infill panels, it is better not to rely on infill panels to provide lateral stability.

The lateral loads acting on each shear/core wall should be obtained from a global analysis of the structure, taking into account wall-frame interaction, coupling effects and eccentricities of the applied loads. The term “shear centre” has been referred to in the code, but in reality, due to warping restraints and coupling effects, the shear centre of each shear/core wall varies in position with height and is in general very difficult to define. Fortunately, most of the existing computer analysis methods do not rely on the concept of shear centre and therefore there is no need to determine the position of the shear centre any more.

As mentioned several times before, although shear walls are generally designed to carry in-plane loads only, they may be subjected to out-of-plane loads due to the application of accidental loading (a requirement for robustness design) and probably also due to the second order effects arising from out-of-plane deflection of the walls and the axial loads acting on the walls.

## 5.5 Transfer structures

Transfer structures are horizontal elements, which transfer vertical loads from vertical elements above laterally to other vertical elements below at different positions. They are necessary wherever there are discontinuities in the vertical elements. In the process of transferring the vertical loads laterally, the transfer structures are themselves subjected to very large vertical shear loads and thus have to be quite deep or thick.

Because of the large stiffness, a transfer structure is quite sensitive to the differential axial shortening of the vertical elements supporting it. Hence, the vertical loads should be transferred through the transfer structure to the vertical elements below as uniformly distributed as possible. Where the interaction between the transfer structure and the vertical elements below could be significant, the transfer structure should be analysed together with the vertical elements below as an integral structure. If there are shear walls above the transfer structure, the local effects of the shear walls should also be considered.

Controlling the relative lateral deflection $< {}^{H_s}/_{700}$ at the transfer structure level with respect to the storey below avoids abrupt reduction in stiffness at the storey below the transfer structure.

When horizontal loads are transferred through the transfer structure, the vertical elements above and below would be subjected to horizontal shear loads and would therefore deflect laterally causing the development of large concentrated moments at the joints between the vertical elements and the transfer structure. Such concentrated moments should be considered in the design. Moreover, the possible second order effects on the vertical elements due to sidesway of the transfer structure should also be considered.

## 5.6 Precast elements

There is no additional guidance given in the code for the design of precast elements. Basically, precast elements should be designed to the same standard as the in-situ cast elements. If the construction sequence and schedule could have significant effects on the behaviour of the structure, such effects should be fully allowed for in the analysis of structure.

# 6 ULTIMATE LIMIT STATES

## 6.1 Members in flexure

### 6.1.1 General

This section deals with the design of beams and various types of slabs. The general requirements for the design of beams are given in Clause 6.1.2. These requirements are applicable also to slabs. The additional requirements for slabs are given in Clauses 6.1.3, 6.1.4 and 6.1.5.

### 6.1.2 Beams

The provisions given in the code are for the design of beams of normal proportions only. Deep beams (span/overall depth ratio ≤ 2 for simply-supported beams and span/overall depth ratio ≤ 2.5 for continuous beams) have been excluded entirely. For the design of deep beams, specialist literature should be consulted. Readers may refer to the following design references for details: 1.) Ove Arup & Partners, The Design of Deep Beams in Reinforced Concrete (G2D), CIRIA, 1977, 132 pp., 2.) Goodchild, C.H., Morrison, J. and Vollum, R.L., Strut-and-tie Models – How to Design Concrete Members Using Strut-and-Tie Models in accordance with Eurocode 2, MPA The Concrete Centre, 2014, 64pp.

To maintain lateral stability (i.e. to avoid lateral buckling of the compression zone of the beam), the following slenderness limits need to be imposed:

- for simply-supported and continuous beams, the clear distance between lateral restraints should not exceed the minimum of ($60b_c$ or ${250b_c}^2/d$);
- for cantilever beams with lateral restraint only at support, the distance between the lateral restraint and the free end should not exceed the minimum of ($25b_c$ or ${100b_c}^2/d$);

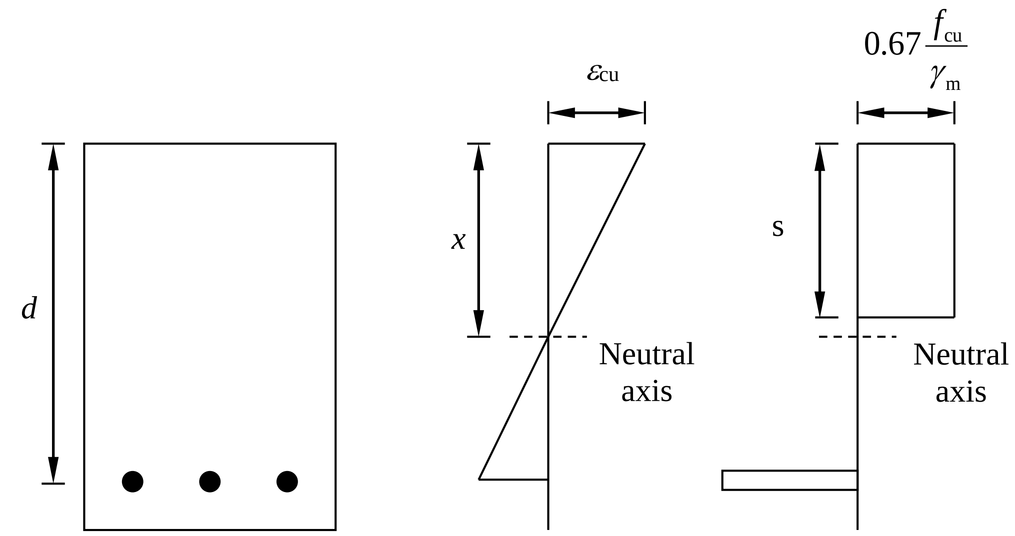
in which $b_c$ is the breadth of the compression zone and $d$ is the effective depth.

<u>The provisions for the evaluation of the *design resistance moment* of beams are given in the following</u>:

*Assumptions:*

- The strain distribution may be evaluated based on the “plane sections remain plane after bending” assumption.
- The stresses in the concrete in compression may be derived using the stress-strain curve given in Section 3.1 of the code with $\gamma_m = 1.5$. Alternatively, the simplified stress block given in Figure 6.1 may be used.
- The tensile strength of concrete is negligible.
- The stresses in the steel reinforcement may be derived using the stress-strain curve given in Section 3.2 of the code with $\gamma_m = 1.15$.

- The lever arm is not to be taken as greater than 0.95 times the effective depth.
- The effect of any axial load may be ignored if it does not exceed $0.1f_{cu}$ times the cross-sectional area.




Section Strain Stress

Note:

$\varepsilon_{cu} = 0.0035$ for $f_{cu} \leq 60\,\text{N/mm}^2$

$\varepsilon_{cu} = 0.0035 - 0.00006\sqrt{(f_{cu} - 60)}$ for $f_{cu} > 60\,\text{N/mm}^2$

$s = 0.9x$ for $f_{cu} \leq 45\,\text{N/mm}^2$

$s = 0.8x$ for $45 < f_{cu} \leq 70\,\text{N/mm}^2$

$s = 0.72x$ for $70 < f_{cu} \leq 100\,\text{N/mm}^2$

Figure 6.1 Simplified stress block for concrete at ULS

In BS8110: Part 1, the ultimate concrete strain $\varepsilon_{cu}$ for the flexural design of normal strength concrete is taken as a constant value of 0.0035. Research by both the UK Concrete Society and The University of Hong Kong has shown that the ductility of concrete decreases with higher concrete strength. Hence, the ultimate concrete strain needs to be reduced as the concrete strength increases. The limitation on strain in Figure 6.1 is based on the research by The University of Hong Kong, as published in the following paper:
Ho J.C.M., Kwan A.K.H. and Pam H.J., "Ultimate concrete strain and equivalent rectangular stress block for design of high-strength concrete beams", Structural Engineer, Vol.80, No.16, 2002, pp26-32.

*Limitations:*

- Where the redistribution of moments does not exceed 10%,

  for $f_{cu} \leq 45\,\text{N/mm}^2$, $x/d \leq 0.5$ Equation 6.1

  for $40 < f_{cu} \leq 70\,\text{N/mm}^2$, $x/d \leq 0.4$ Equation 6.2

  for $70 < f_{cu} \leq 100\,\text{N/mm}^2$, $x/d \leq 0.33$ and no moment redistribution

Equation 6.3

- Where the redistribution of moments exceeds 10%,
  for $f_{cu} \leq 45\,\mathrm{N/mm^2}$, $\quad x/d \leq (\beta_b - 0.4)$ Equation 6.4

  for $40 < f_{cu} \leq 70\,\mathrm{N/mm^2}$, $\quad x/d \leq (\beta_b - 0.5)$ Equation 6.5

  in which $\beta_b = \frac{\text{moment after redistribution}}{\text{moment before redistribution}}$ Equation 6.6

BS8110: Part 1 provides a limit to the neutral axis depth to ensure a minimum level of flexural ductility (i.e. that large strains are developed in the tension reinforcement at ULS). For redistribution of moments up to 10%, the limit given is $x/d \leq 0.5$. For redistribution of moments greater than 10% (i.e. $\beta_b < 90\%$), a lower limit of $x/d \leq (\beta_b - 0.4)$ is applied to ensure that sufficiently large strains are developed in the tension reinforcement to provide the plastic hinges with enough rotational capacities for meeting the plastic deformation demand arising from the redistribution of moments. These limits in BS8110 are, however, applicable only to relatively low strength concrete. For concrete of strength higher than 40 N/mm$^2$, which possesses progressively lower ductility as the concrete strength increases, in order to maintain a consistent level of flexural ductility, the tension to balanced steel ratio has to be lowered. Hence, the permissible neutral axis depth has to be reduced accordingly. The University of Hong Kong has carried out some research on this and its recommendations for higher strength concrete are those formulas given above for redistribution of moments up to 10%. For redistribution of moments greater than 10%, the formulas are modified to incorporate $\beta_b$ in a way similar to that of BS8110. More background information on the derivation of these formulas can be found in the following paper:
Ho J.C.M., Kwan A.K.H. and Pam H.J., "Minimum flexural ductility design of high-strength concrete beams", Magazine of Concrete Research, Vol.56, No.1, 2004, pp13-22.

*Design formulas for rectangular beams:*
Two dimensionless factors, namely: *K* and *K'*, are employed in the formulas. They are defined respectively as:

$$K = \frac{M}{bd^2 f_{cu}}$$ Equation 6.7

and

$$K' = \frac{M'}{bd^2 f_{cu}}$$

in which *M* is the required moment of resistance as calculated from the design loads and *M'* is the maximum achievable moment of resistance of the section with no compression reinforcement added and the neutral axis depth $x$ set at the upper limit. If $K < K'$, no compression reinforcement is required, but if $K > K'$, compression reinforcement is required. Figure 6.2 illustrates how *M'* and *K'* are derived. From this figure (Assume $f_{cu} \leq 45$ N/mm$^2$),

$$C = 0.67\frac{f_{cu}}{\gamma_m} \times 0.9bx; \quad z = d - 0.45x$$

$$M' = Cz = 0.67\frac{f_{cu}}{\gamma_m} \times 0.9bx \times (d - 0.45x)$$

$$M' = 0.402\left({}^{x}/_{d}\right)\left(1 - 0.45\,{}^{x}/_{d}\right)bd^2 f_{cu}$$

$$K' = \frac{M'}{bd^2 f_{cu}} = 0.402\left({}^{x}/_{d}\right)\left(1 - 0.45\,{}^{x}/_{d}\right)$$

Note: The depth of stress block is 0.8x ($45 < f_{cu} \leq 70\,\mathrm{N/mm^2}$) and 0.72x ($70 < f_{cu} \leq 100\,\mathrm{N/mm^2}$) for higher strength concrete. The derived equation will be changed accordingly.

Substituting the respective upper limit value of ${}^{x}/_{d}$ into the above equation, the values of $K'$ under different situations are obtained as:

Where the redistribution of moments does not exceed 10%,

for $f_{cu} \leq 45\,\mathrm{N/mm^2}$, $K' = 0.156$

for $45 < f_{cu} \leq 70\,\mathrm{N/mm^2}$, $K' = 0.120$ Equation 6.8

for $70 < f_{cu} \leq 100\,\mathrm{N/mm^2}$, $K' = 0.094$ and no moment redistribution

- Where the redistribution of moments exceeds 10%,

  for $f_{cu} \leq 45\,\mathrm{N/mm^2}$, $K' = 0.402(\beta_b - 0.4) - 0.18(\beta_b - 0.4)^2$

  for $45 < f_{cu} \leq 70\,\mathrm{N/mm^2}$, $K' = 0.357(\beta_b - 0.5) - 0.143(\beta_b - 0.5)^2$

Equation 6.9

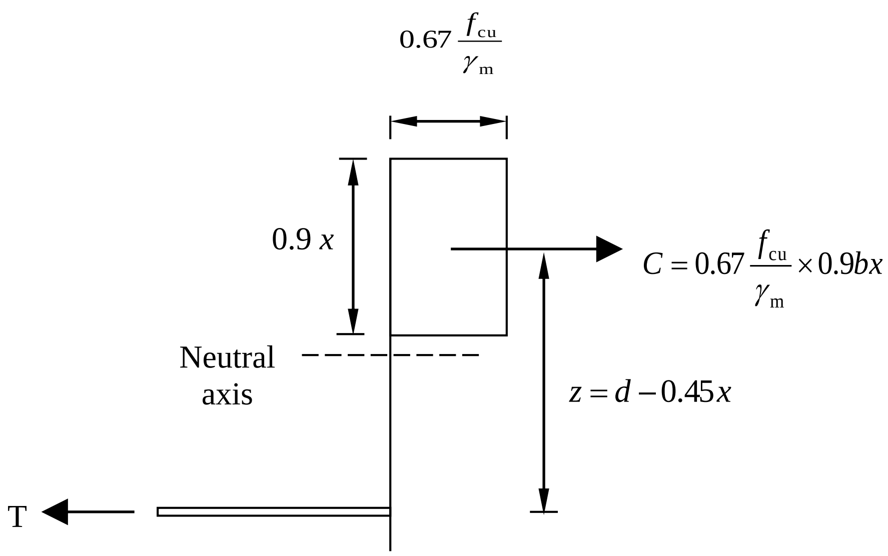



Figure 6.2 Evaluation of *M'* and *K'*

Worked Example 6.1: Design of tension and compression reinforcement for a rectangular section with high strength concrete

A rectangular beam section as shown in figure 6.3 has characteristic material strength of $f_{cu} = 80\,\text{N/mm}^2$ for concrete and $f_y = 500\,\text{N/mm}^2$ for steel. The design sagging moment at the ultimate limit state is 800 kNm. Moment redistribution is not considered.

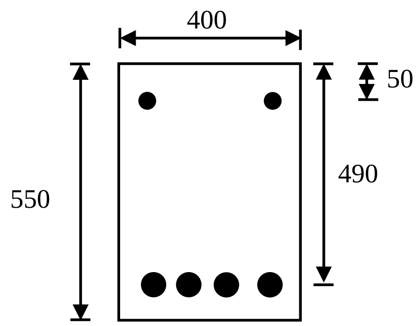



Figure 6.3 Worked example – doubly reinforced concrete beam

For $f_{cu}$ > 70 N/mm$^2$ and no moment redistribution, $K$' = 0.094 Equation 6.8

$$K = \frac{M}{bd^2 f_{cu}} = \frac{800\times10^6}{(400)(490^2)(80)} = 0.104 > 0.094$$ Equation 6.7

Compression reinforcement is required.

$$z = d\left(0.5 + \sqrt{0.25 - \frac{K'}{0.9}}\right) = 490\left(0.5 + \sqrt{0.25 - \frac{0.094}{0.9}}\right) = 432 \text{ mm}$$

Equation 6.13

Check the status of compression steel,

$$\frac{d'}{x} = \frac{50}{\frac{d-z}{0.36}} = \frac{50}{\frac{490-432}{0.36}} = 0.31$$

$$< 1 - \frac{2.175\times10^{-3}}{\varepsilon_{cu}} = 0.327 \text{ (Compression steel will have yielded)}$$

$$A_s' = \frac{(K-K')f_{cu}b_c d^2}{0.87 f_y (d-d')} = \frac{(0.104-0.094)(80)(400)(490)^2}{0.87(500)(490-50)} = 401 \text{ mm}^2$$ Equation 6.15

Minimum amount of reinforcement $= 0.2\% A_c = 330\text{mm}^2$ Table 9.1

Provide 2T16 at top, $A_s = 402\text{mm}^2$

$$A_s = \frac{K' f_{cu} b_c d^2}{0.87 f_y z} + A'_s = \frac{0.094(80)(400)(490)^2}{0.87(500)(432)} + 401 = 4244 \ \ \text{mm}^2$$ Equation 6.16

Provide 4T40, $A_s = 5024\text{mm}^2$

$$\rho_s = \frac{5024}{400 \times 550} = 2.3\% > 0.13\% \ \text{ and } < 4\%$$ Table 9.1

Worked Example 6.2: Design of RC rectangular beam with moment redistribution

A RC beam with rectangular cross-section, 400 mm (width) × 600 mm (depth), has characteristic material strength of $f_{cu} = 50\,\text{N/mm}^2$ for concrete and $f_y = 500\,\text{N/mm}^2$ for steel. The design moment at ultimate limit state is 750 kNm. A moment distribution of 20% is considered.

$$\beta_b = \frac{\text{Moment after distribution}}{\text{Moment before distribution}} = 0.8$$ Equation 6.6

The effective depth is taken as 540 mm.

Limit of neutral axis depth is given as

$$x \leq (\beta_b - 0.5)d = (0.8 - 0.5)(540) = 162 \ \ \text{mm}$$ Equation 6.5

$$x = \frac{d - z}{0.4}$$

$$\rightarrow z = d - 0.4x = 540 - 0.4(162) = 475 \ \ \text{mm}$$ Equation 6.11

Moment of resistance of signly reinforced beam

$$M_c = 0.45 f_{cu} b (0.8x) z = 0.45(50)(400)(0.8 \times 162)(475)$$
$$= 554 \ \ kNm < 750 \ \ kNm$$

Compression reinforcement is required.

Check the status of compression steel,

$$\frac{d'}{x} = \frac{60}{162} = 0.37 \quad < 0.38 \ \text{ (Compression steel will have yielded)}$$

$$A'_s = \frac{(M - M_c)}{0.87 f_y (d - d')} = \frac{(750 - 554) \times 10^6}{0.87(500)(540 - 60)} = 939 \ \ \text{mm}^2$$

Minimum amount of reinforcement $= 0.2\% A_c = 480 \ \ \text{mm}^2$ Table 9.1

Provide 3T20 at top, $A_s = 942\text{mm}^2$

$$A_s = \frac{M_c}{0.87 f_y z} + A'_s = \frac{554 \times 10^6}{0.87(500)(475)} + 939 = 3620 \ \ \text{mm}^2$$

Provide 3T40, $A_s = 3768 \text{ mm}^2$

In case of without moment redistribution

$$K = \frac{M}{bd^2 f_{cu}} = \frac{750\times10^6}{(400)(540^2)(50)} = 0.129 < 0.120 \qquad \text{Equation 6.7}$$

Compression is required.

$$z = d\left(0.5 + \sqrt{0.25 - \frac{K'}{0.9}}\right) = 540\left(0.5 + \sqrt{0.25 - \frac{0.12}{0.9}}\right) = 454 \text{ mm}$$

Equation 6.13

Check the status of compression steel,

$$\frac{d'}{x} = \frac{60}{\frac{d-z}{0.4}} = \frac{60}{\frac{540-454}{0.4}} = 0.28 < 0.38 \text{ (Compression steel will have yielded)}$$

$$A'_s = \frac{(K-K')f_{cu}b_c d^2}{0.87 f_y (d-d')} = \frac{(0.129-0.120)(50)(400)(540)^2}{0.87(500)(540-60)} = 251 \text{ mm}^2 \qquad \text{Equation 6.7 \& 6.8}$$

Minimum amount of reinforcement $= 0.2\% A_c = 480 \text{ mm}^2$ Table 9.1

Provide 3T16 at top, $A_s = 603 \text{mm}^2$

$$A_s = \frac{K' f_{cu} b_c d^2}{0.87 f_y z} + A'_s = \frac{0.12(50)(400)(540)^2}{0.87(500)(454)} + 251 = 3795 \text{ mm}^2 \qquad \text{Equation 6.16}$$

Provide 2T40 + 2T32, $A_s = 4122 \text{mm}^2$

| | **With moment redistribution** | **Without moment redistribution** |
|---|---|---|
| **Compression steel** | 939 mm$^2$ | 480 mm$^2$ |
| **Tension steel** | 3620 mm$^2$ | 3795 mm$^2$ |

*Design formulas for flanged beams:*
If the neutral axis lies within the flange, the design may be carried out following the same procedures as for rectangular beams. Otherwise, the design should be carried out by direct application of the assumptions given in Clause 6.1.2.4(a).

The provisions for the evaluation of the *design shear resistance* of beams are given in the following:

*Shear stress*:
The design shear stress $v$ at any section should be calculated from:

$$v = \frac{V}{b_v d} \qquad \text{Equation 6.19}$$

where $V$ is design shear load and $b_v$ is the breadth of the section (or the breadth of the web or rib below the flange in case of a flanged beam).

In no case should $v$ exceed $0.8\sqrt{f_{cu}}$ or 7.0 N/mm$^2$, whichever is the lesser, regardless of the amount and type of shear reinforcement provided. This is due to the assumption of truss analogy and to prevent the diagonal compressive stresses to cause crushing of the web concrete. It should be noted that BS8110: Part 1 limits the design shear stress to only 5.0 N/mm$^2$. Some tests on the shear capacity of high-strength concrete beams have been carried out at The University of Hong Kong. The test results revealed that the shear strength of a reinforced concrete beam without stirrups increases with increasing concrete compressive strength but the rate of increase in shear strength gradually decreases as the concrete strength increases until the shear strength stops increasing at a cube strength of about 80 N/mm$^2$. Based on this, the limiting value of design shear stress has been increased to $0.8\sqrt{f_{cu}}$ with $f_{cu}$ set at 80 N/mm$^2$. This actually gives a limiting value of design shear stress of 7.16 N/mm$^2$, but has been rounded down to 7.0 N/mm$^2$. However, at such high design shear stress level, very heavy shear reinforcement would be required. Moreover, the compression struts formed inside the beam would be subjected to fairly high compressive forces. To avoid brittle failure, it should be prudent to provide generous lateral confinement to the compression struts. Particular attention should also be paid to the use of good quality aggregate because the shear strength of high-strength concrete is quite sensitive to the quality of aggregate used. The afore-mentioned tests carried out at The University of Hong Kong have been published in:
Islam M.S., Pam H.J. and Kwan A.K.H., "Shear capacity of high-strength concrete beams with point of inflection within shear span", Proceedings, Institution of Civil Engineers, Structures and Buildings, Vol.128, February, 1998, pp91-99.

*Shear reinforcement:*
The shear reinforcement to be provided is dependent mainly on the direction and the magnitude of $v$ relative to $v_c$ (the design concrete shear stress without shear reinforcement) and $v_r$ (the nominal shear strength to be provided by minimum shear reinforcement). $v_c$ is as given in Table 6.3 of the code, while $v_r$ is to ensure that when shear cracking occurs, the tension originally carried by the concrete before cracking can be transferred to the shear reinforcement without causing it to yield. In BS8110: Part 1, $v_r$ is set at a constant value of 0.4 N/mm$^2$. However, as the concrete strength increases, the shear force at which cracking occurs increases. Hence, the tension, which has to be transmitted to the shear reinforcement when shear cracking occurs, also increases and the area of shear reinforcement has to be increased to prevent it from yielding at the time of shear cracking. The required minimum shear reinforcement is proportional to the tensile strength of concrete, which is assumed to be proportional to its compressive strength to the power $^2/_3$. Therefore, for concrete strength higher than 40

N/mm$^2$, the minimum shear reinforcement has to be increased by a factor of $\left({}^{f_{cu}}\!/_{40}\right)^{2/3}$. The amount of shear reinforcement to be provided is given by:

Where $v < 0.5v_c$ throughout the beam:

no requirement for structural elements of minor importance;
minimum links for structural elements of importance.

Where $0.5v_c \leq v < (v_c + v_r)$:

minimum links to be provided.

Where $(v_c + v_r) \leq v < \text{min.}\left(0.8\sqrt{f_{cu}} \text{ or } 7.0\ \text{N/mm}^2\right)$:

$A_{sv} \geq {}^{b_v s_v (v - v_c)}\!/_{0.87 f_{yv}}$, if only links provided;

up to 50% of the shear resistance to be provided by the shear reinforcement may be in the form of bent-up bars.

*Concrete shear stress*:

The design concrete shear stress $v_c$ is derived from the expression:

$$v_c = 0.79 \left(\frac{100A_s}{b_v d}\right)^{1/3} \left(\frac{400}{d}\right)^{1/4} \left(\frac{f_{cu}}{25}\right)^{1/3} \frac{1}{\gamma_m} \qquad \text{Table 6.3}$$

in which the value of $f_{cu}$ should not be taken as greater than 80 N/mm$^2$ (note that BS8110: Part 1 limits the value of $f_{cu}$ at not greater than 40 N/mm$^2$). The equation is based on an extensive study of test data, as shown in Figure 6.4. The term $A_s$ is the area of longitudinal tension reinforcement, which continues for a distance of at least *d* beyond the section being considered. At supports, only the part of longitudinal tension reinforcement, which meets the curtailment and anchorage requirements at the section being considered, should be counted towards $A_s$.

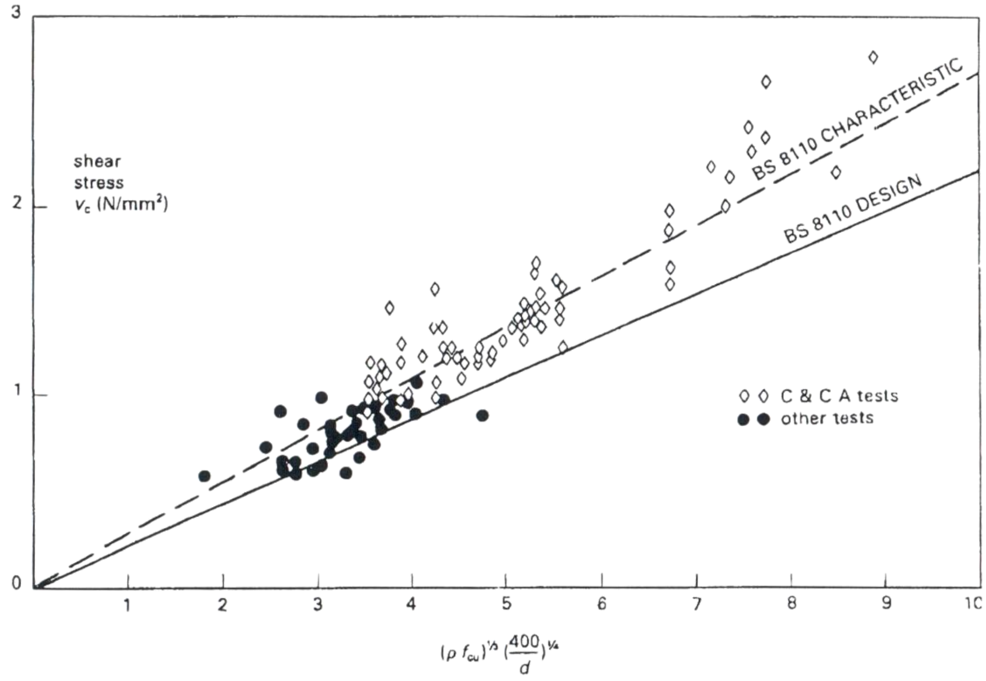



Figure 6.4 Shear strength of beams without shear reinforcement

(Adapted from Rowe, R.E., Handbook to British Standard BS 8110:1985 – Structural Use of Concrete, London, England: Palladian Publications, 1987, 206pp)

*Shear resistance under different conditions*:
The provisions given in the code for the determination of shear resistance under different conditions are similar to those in BS8110: Part 1: 1997 except that the material safety factor of the shear reinforcement is set at 1.15 and when $f_{cu} > 40\,\mathrm{N/mm^2}$, both the design concrete shear stress $v_c$ and the limiting value of design shear stress $v$ are higher (increase in shear strength until $f_{cu}$ reaches 80 $\mathrm{N/mm^2}$ is allowed).

*Enhanced shear strength near supports*
At a distance d from the support, Figure 6.4 shows that the capacity of the section is increasing very rapidly. The enhancement of shear strength may be taken into account in the design of sections near a support by increasing the design concrete stress $v_c$ to ${2dv_c}/{a_v}$. $a_v$ is distance measured from the support to the section being designed.

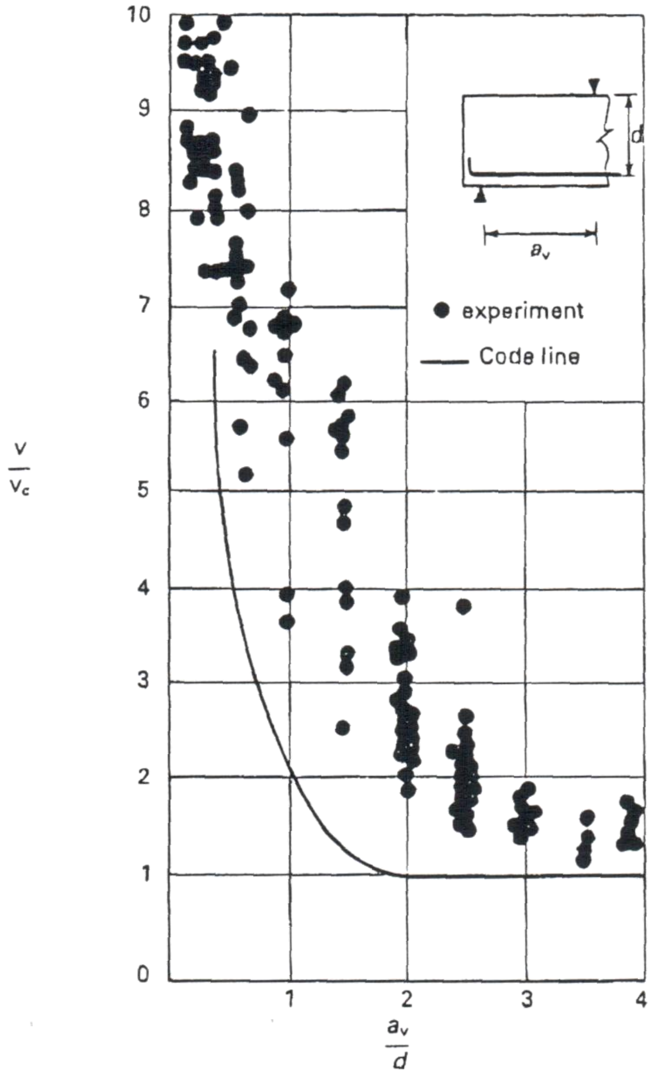



Figure 6.5: Ultimate shear stresses for beams loaded close to supports

(Adapted from Rowe, R.E., Handbook to British Standard BS 8110:1985 – Structural Use of Concrete, London, England: Palladian Publications, 1987, 206pp)

Worked Example 6.3: Design of RC flanged beam

A simply supported beam has a flanged section as shown as shown in Figure 6.6

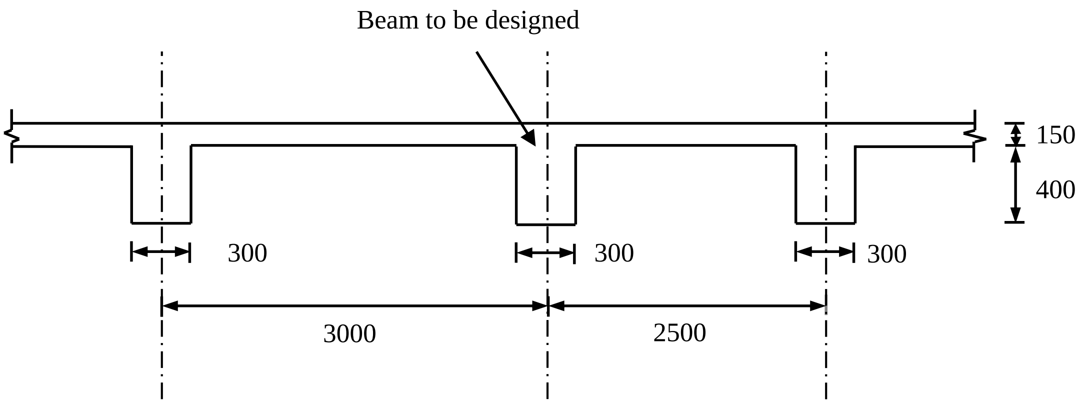



Figure 6.6 Worked example – flanged beam

The beam has characteristic material strength of $f_{cu} = 40\,\mathrm{N/mm^2}$ for concrete and $f_y = 500\,\mathrm{N/mm^2}$ for steel. The ultimate design uniformly distributed load is $w_u$ = 60 kN/m$^2$ The distance between points of zero moment $l_p$ is 6 m.

**Effective width**

$b_1 = 0.5(3000 - 300) = 1350$ mm

$b_2 = 0.5(2500 - 300) = 1100$ mm

$l_p = 6$m — Clause 5.2.1.2

$b_{\mathrm{eff},1} = \min[0.2b_1 + 0.1l_p, 0.2l_p, b_1]$ — Equation 5.2

$= \min[0.2(1350) + 0.1(6000) = 870$ mm,

$0.2(6000) = 1200$ mm, $1350$ mm]

$= 870$ mm

$b_{\mathrm{eff},2} = \min[0.2b_2 + 0.1l_p, 0.2l_p, b_2]$ — Equation 5.2

$= \min[0.2(1100) + 0.1(6000) = 820$ mm,

$0.2(6000) = 1200$ mm, 1100 mm]

$= 820$ mm

$b_{\mathrm{eff}} = 300 + 870 + 820 = 1990$ mm — Equation 5.1

**Design for bending**

The effective depth is taken as 490 mm.

For $f_{cu} \leq 40$ N/mm$^2$ and no moment redistribution, $K' = 0.156$

$$K = \frac{M}{bd^2 f_{cu}} = \frac{270\times10^6}{(1990)(490^2)(40)} = 0.0141 < 0.156$$ Equation 6.7

$$z = 490\left(0.5 + \sqrt{0.25 - \frac{0.0141}{0.9}}\right) = 482.2 \text{ mm} > 0.95d = 465.5 \text{ mm}$$

Equation 6.10

Takes $z = 465.5$mm

Depth of neutral axis

$$x = \frac{d-z}{0.45} = \frac{490-465.5}{0.45} = 54.4 \text{ mm} < 150 \text{ mm}$$ Equation 6.11

Thus, the stress block lies within the flange. The design of flanged section can be treated as the design of rectangular section.

$$A_s = \frac{270\times10^6}{0.87(500)(465.5)} = 1334 \text{ mm}^2$$ Equation 6.12

Provide 3T25 at bottom, $A_s$ = 1471 mm$^2$.

$$\rho_s = 1471/(300 \times 550) = 0.98\% > 0.18\%$$ Table 9.1

**Shear stress between the web and flange**

Since $b_{\text{eff,i}} > 0.1 l_p = 600$ mm, check for shear stress between the web and flange is required.

Clause 5.2.1.2(a)

Takes $\Delta x = L/4 = 1.5$ m (half of the distance between locations with zero and maximum bending moment) EC2, clause 6.2.4

The change in moment ΔM over distance Δx = L/4 from the zero moment is

$$\Delta M = \frac{w_u L}{2}\times\frac{L}{4} - \frac{w_u L}{2}\times\frac{L}{8} = \frac{3 w_u L^2}{32} = \frac{3\times60\times6^2}{32} = 202.5 \ kNm$$

The change in logitudinal force ΔF at the flange-web interface is

$$\Delta F_d = \frac{\Delta M}{(d-h_f/2)}\times\frac{b_{eff0}}{b_{eff}} = \frac{202.5\times10^3}{(490-150/2)}\times\frac{870}{1990} = 235.5 \ kN$$

Shear stress

$$v_{sf} = \frac{\Delta F_d}{\Delta x h_f} = \frac{235.5 \times 10^3}{1500 \times 150} = 1.06 \ \text{N/mm}^2$$

$$\frac{A_{sf}}{s_f} \geq \frac{v_{sf} h_f}{0.87 f_y}$$

$$\rightarrow \frac{A_{sf}}{s_f} \geq \frac{1.06 \times 150}{0.87 \times 500} = 0.37$$

Use T12-300 c.c

### 6.1.3 Solid slabs supported by beams or walls

The provisions for the design of beams are also applicable to solid slabs.

Slabs subjected to distributed and concentrated loads may be analysed using an appropriate elastic analysis method or alternatively a plastic analysis method. For elastic analysis, an analytical method may be used if the slab has simple geometry but the finite element method may have to be used if the slab has complex geometry or is subjected to a complicated arrangement of loading. For plastic analysis, both Johansen's yield line method and Hillerborg's strip method are considered acceptable.

For slabs subjected to concentrated loads, there will be concentrated moment occurring at each loading point. Due to stress concentration, the concentrated moment could be many times larger than the average moment acting on the section underneath the loading point parallel to the nearest support. To capture such concentrated moment, if the finite element method of analysis is employed, a very fine mesh is required. However, in reality, because of the large magnitude of the concentrated moment, the section at the loading point would yield at an early stage and the moment would be redistributed to other parts of the slab, resulting in a less concentrated and more uniform distribution of moment. As slabs are generally quite lightly reinforced and should therefore possess a fair amount of flexural ductility, such redistribution of moments could be very substantial. To take advantage of the redistribution of moments, which can help to reduce the maximum moment to be designed for, it may be assumed that the slab moment is resisted by an effective width over which the total moment (the total moment may be obtained by integrating the moment per unit width determined by a rigorous elastic analysis along the effective width) is uniformly distributed. For a slab simply supported on two opposite edges, the code has provided guidelines for the evaluation of the effective width. These guidelines are the same as those given in BS8110: Part 1: 1997. For any other slab with a more complicated geometry or support arrangement, no general guideline could be given and rigorous limited ductility elasto-plastic analysis will have to be carried out if advantage is to be taken of the possible redistribution of moments to reduce the maximum moment to be designed for.

Apart from the above redistribution of moments across the effective width, the redistribution of moments from the supports to the mid-span sections may also be carried out as per Clause 5.2.9 of the code. If the analysis of the slab is carried out for the single load case of all spans loaded in accordance with Clause 6.1.3.2(c), the redistribution of moments is limited to 20%.

For the elastic analysis of rectangular two-way spanning slabs subjected to uniformly distributed loads, the code has provided formulas for the direct evaluation of the design maximum moments. These formulas are identical to those given in BS8110: Part 1: 1997.

Regarding the shear design of solid slabs, the shear resistance of a solid slab is to be evaluated as if the solid slab is a beam (i.e. the values of $v_c$ and $v_r$ are to be evaluated in the same way) but the shear reinforcement may be designed according to the following more relaxed rules:

Where $v < v_c$: Table 6.8

no requirement for shear reinforcement.

Where $v_c \leq v < (v_c + v_r)$:

minimum links to be provided.

Where $(v_c + v_r) \leq v < \min.\left(0.8\sqrt{f_{cu}} \text{ or } 7.0\ \mathrm{N/mm^2}\right)$:

$A_{sv} \geq {b_v s_v (v - v_c)}\big/{0.87 f_{yv}}$, if only links provided;

the shear reinforcement may be provided in the form of links and/or bent-up bars in any combination.

For industrial on-grade slab design, it is not mentioned in the code of practice. Readers may refer to a design reference for details: Technical Report 34: Concrete Industrial Ground Floors, The Concrete Society, 2016, 91pp.

### 6.1.4 Ribbed slabs

There are two types of ribbed slabs: one type with the topping considered to contribute to structural strength and the other type with the topping not considered to contribute to structural strength. Each of them has to meet certain requirements as stipulated in Clause 6.1.4.1 of the code. Where the topping is considered to contribute to structural strength, its thickness should not be less than the minimum thickness given in Table 6.9.

The continuous span of a ribbed slab may be analysed as a one-way spanning solid slab as per Clause 6.1.3.2 of the code. If the ribbed slab has equal structural properties in two perpendicular directions (see Clause 6.1.4.2) and its flanges and ribs have sufficient torsional stiffness (see Clause 5.2.1.1), it may be analysed as a two-way spanning solid slab in accordance with Clause 6.1.3.3 or as a flat slab in accordance with Clause 6.1.5, whichever is more appropriate.

If it is impracticable to provide sufficient reinforcement to develop the full design moments at the supports, the design moments at the supports may be redistributed to the mid-span sections. Clause 6.1.4.2 of the code allows 100% of the moments at the supports to be redistributed to the mid-span sections so that effectively the slabs are designed as a series of simply supported spans. If this is done, sufficient reinforcement should be provided over the supports to control cracking. The code recommends that such reinforcement should have an area of not less than 25% of that in the middle of the adjoining spans and should extend at least 15% of the spans into the adjoining spans. Nevertheless, the author is of the view that in order to avoid excessive cracking, the redistribution of moments should be limited to say only 50%; 100% redistribution of moments is a bit too much and may adversely affect the serviceability of the structure.

Regarding the design resistance moment, the provisions given in Clause 6.1.2.4 for determining the design resistance moment of beams may also be applied to ribbed slabs. When determining the design resistance moment of ribbed slabs, the strength contribution of the forming blocks, which remain part of the completed structure, may be taken into account.

Regarding the design shear resistance, the provisions given in Clause 6.1.2.5 for determining the design shear resistance of flanged beams may also be applied to ribbed slabs, except that the breadth of the rib may be increased to allow for the contribution of the forming blocks. The shear reinforcement may be design as in the case of solid slabs.

### 6.1.5 Flat slabs

The provisions given in the code, which are identical to those given in BS8110: Part 1: 1997, are for application to flat slabs supported by a generally rectangular arrangement of columns with a longer to shorter spans ratio not exceeding 2.0. For such kind of flat slabs, the equivalent frame method may be used for the analysis, as provided for in the code. For flat slabs with more complicated arrangement of columns, other methods, e.g. the finite element method, may have to be used, but in such cases the design engineer will have to make his/her own judgement as to whether the provisions in this section are applicable. Yield line method provides the most economical solution and the most suitable arrangement of reinforcement for working load conditions with consequent implication for cracking and deflection.

In any direction, the effective dimension of a column head $l_h$ may be taken as:

$$l_h = \min.(i_{ho} \text{ or } l_{h\max}) \qquad \text{Clause 6.1.5.1 (b)}$$

where $i_{ho}$ is the actual dimension measured 40 mm below the soffit of the slab or drop, and $l_{h\max}$ is given by:

$$l_{h\max} = l_c + 2(d_h - 40) \qquad \text{Equation 6.37}$$

in which $l_c$ is the dimension of column in same direction and $d_h$ is the depth of the column head.

Drops are provided mainly to increase the punching shear resistance of the slab. Their possible influences on the distribution of moments within the slab should be ignored unless the smaller dimension of the drop is at least 1/3 of the smaller dimension of the surrounding panels. When checking the shear resistance, two critical perimeters should be considered. $1.5d_d$ from the face of the column; $1.5d_s$ from the outer edge of the drop. ($d_d$ is the effective depth of drop; $d_s$ is the effective depth of slab.)

For flat slabs consisting of a series of rectangular panels, i.e. supported by a generally rectangular arrangement of columns, the equivalent frame method of dividing the structure longitudinally and transversely into frames comprising of columns and strips of slab may be used for the analysis. Although for analysing the equivalent frames, simplified methods of further dividing each frame into sub-frames as mentioned in Clause 6.1.5.2(d) of the code are permitted, it is considered better to employ a more rigorous analysis method based on either grillage analysis or finite element analysis.

In the use of the finite element method for analysis of slabs, the results at a load case obtained at each point of a slab is the general "moment field" comprising not only bending moments in 2 orthogonal directions (*x* and *y*), but also a "twisting moment" with symbols ($M_x$, $M_y$, $M_{xy}$). The phenomenon is completely analogous to that of a "stress field" comprising 2 orthogonal orientated direct stresses and a shear stress, i.e. $(\sigma_x, \sigma_y, \tau_{xy})$. Structural design has to take the "twisting moment" into account. Wood (1968) has derived the following Wood Armer Equations for determination of the design moments in *x* and *y* direction that can cover the effect of the twisting moment. The equations works on determining design bending moments at *x* and *y* directions $M_x{}^*$ and $M_y{}^*$ that can cover bending moments generated by $M_x$, $M_y$, $M_{xy}$ in all directions. As similar to that of the direct stress problem, the bending moment at angle $\theta$ to the *x*-axis generated by the moment field is

$$M_x \cos^2\theta + M_y \sin^2\theta + 2\,M_{xy}\cos\theta\sin\theta$$

The equations also work on the Johansen's Yield Criterion which gives moment of resistance of the slab structure at an angle $\theta$ to the *x* axis by $M_x{}^*$ and $M_y{}^*$ as

$$M_x{}^*\cos^2\theta + M_y{}^*\sin^2\theta$$

So the criterion for design is :

$$M_x{}^*\cos^2\theta + M_y{}^*\sin^2\theta \geq M_x \cos^2\theta + M_y \sin^2\theta + 2\,M_{xy}\cos\theta\sin\theta$$

Based on the above equation, the optimal solutions of $M_x{}^*$ and $M_y{}^*$ are searched for various cases and the Wood Armer Equations are derived. For these equations, sagging moments carry positive sign and hogging moments carry negative sign and

$M_x{}^*$ and $M_y{}^*$ are the design moments. The designer will design the plate structure as having purely these bending moments.
For sagging moment,
Generally $M_x{}^* = M_x + |M_{xy}|$; $M_y{}^* = M_y + |M_{xy}|$
If either $M_x{}^*$ or $M_y{}^*$ is found to be negative, then such a value is changed to zero as :
$M_x{}^* = M_x + \left|\frac{M_{xy}{}^2}{M_y}\right|$ with $M_y{}^* = 0$ or $M_y{}^* = M_y + \left|\frac{M_{xy}{}^2}{M_x}\right|$ with $M_x{}^* = 0$;

Similarly for hogging moment
Generally $M_x{}^* = M_x - |M_{xy}|$; $M_y{}^* = M_y - |M_{xy}|$
If either $M_x{}^*$ or $M_y{}^*$ is found to be positive, then such a value is changed to zero as :
$M_x{}^* = M_x - \left|\frac{M_{xy}{}^2}{M_y}\right|$ with $M_y{}^* = 0$ or $M_y{}^* = M_y - \left|\frac{M_{xy}{}^2}{M_x}\right|$ with $M_x{}^* = 0$;

Some structural design software has incorporated the Wood's Equations for structural design.

For equivalent frame analysis, the slab panels should be assumed to be divided into column strips and middle strips, as depicted in Figure 6.6 of the code. After obtaining the design moments from the analysis, the design moments should be apportioned between the column strips and middle strips as per the ratios given in Table 6.10 of the code.

Rules for dealing with the positive and negative design moments in internal and edge panels and for dealing with openings in panels are given in Clauses 6.1.5.3, 6.1.5.4 and 6.1.5.5 of the code, respectively. These are the same as those given in BS8110: Part 1: 1997.

The critical consideration for shear is the punching shear around the columns. To design for punching shear at a column, the design effective shear force $V_{\text{eff}}$ needs first to be determined, as per the code using Equation 6.40 for an internal column or Equation 6.41 for a corner or edge column. It should be noted that the design effective shear force $V_{\text{eff}}$ is not the same as the design shear transferred to the column and actually comprises of two components:

- the shear force component arising from the design shear $V_t$ transferred to the column; and
- the shear force component arising from the design moment $M_t$ transferred to the column.

The above two components are more apparent if Equation 6.40 and Equation 6.41 are rewritten into the following forms:

$V_{\text{eff}} = V_t + 1.5\frac{M_t}{x_{sp}}$ for an internal column Equation 6.40

$V_{\text{eff}} = 1.25V_t + 1.5\frac{M_t}{x_{sp}}$ for a corner or edge column Equation 6.41

The maximum design shear stress at a column so evaluated using Equation 6.40 or Equation 6.41 above and Equation 6.42 on a perimeter bounding the column face or the column head should not exceed $0.8\sqrt{f_{cu}}$ or 7.0 N/mm$^2$, whichever is the lesser (note the increase in maximum design shear stress compared to BS8110: Part 1: 1997). The value of the effective shear force when using these coefficients may be determined from the simplified factors 1.15 for internal columns, 1.25 for corner columns and edge columns bent about an axis parallel to the free edge and 1.4 for edge columns bent about an axis perpendicular to the free edge.

The nominal design shear stress $v$ appropriate to a particular perimeter being considered is calculated as:

$$v = \frac{V}{ud} \qquad \text{Equation 6.43}$$

where $u$ is the effective length of the outer perimeter of the punching failure zone. Based on the value of $v$ relative to $v_c$, the shear reinforcement may be designed according to the following rules (which are similar but not exactly the same as those for solid slabs):

Where $v < v_c$:
no requirement for shear reinforcement.

Where $v_c \leq v \leq 2v_c$:
shear reinforcement may be provided in the form of links, as per Equation 6.44 when $v_c \leq v \leq 1.6v_c$ and Equation 6.45 when $1.6v_c < v \leq 2v_c$;
the shear reinforcement provided should not be less than that required to provide a nominal shear strength of 0.4 N/mm$^2$.

Where $2v_c < v$ and a reinforcing system is provided:
justification should be provided to demonstrate the validity of the design.

Worked Example 6.4: Design for flab slab

Design an interior flat slab with columns at 6.5 m centres in each direction and supports an imposed load of $4\ \mathrm{kN/m^2}$ as shown in Figure 6.7. The effective column heads are to be made 1.4 m diameter. The thickness of the flat slab is 250 mm and the drop panel is 250 mm square by 100 mm deep. The cover to the reinforcement is 25 mm and the preferred bar size is 12 mm. The characteristics strength of materials are $f_{cu}$ = 30 N/mm$^2$ and $f_y$ = 500 N/mm$^2$, repsectively.

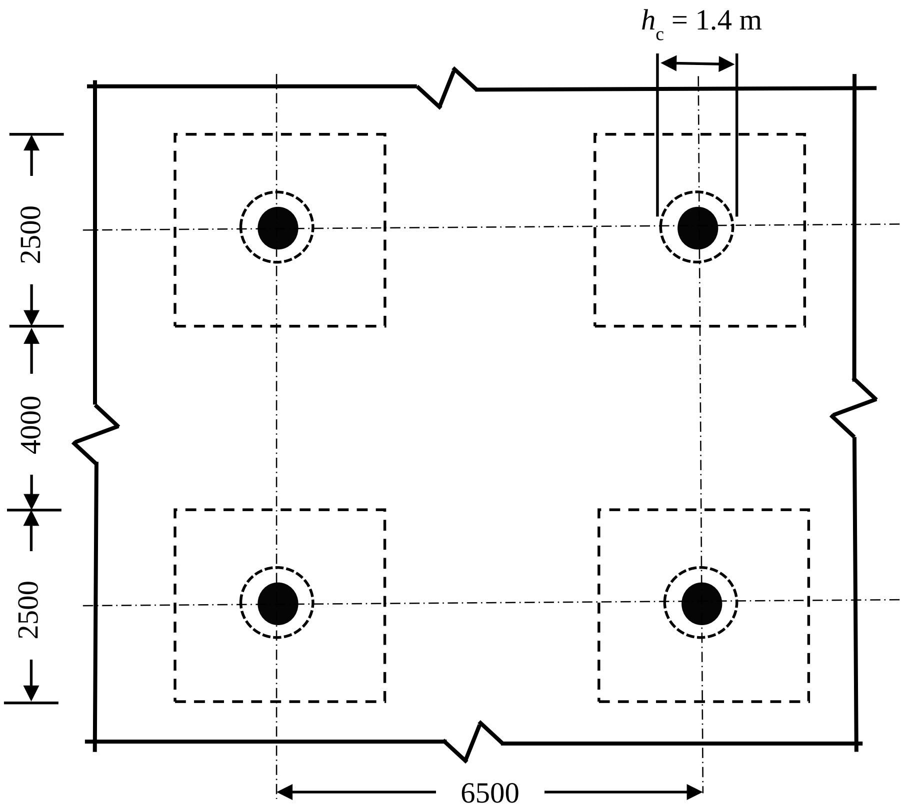



Figure 6.7 Worked example – flat slab

Design Load of Flat Slab

*Permanent Load*

Self-weight of slab $= 0.25 \times 24.5 \times 6.5^2 = 259$ kN

Self-weight of drop $= 0.1 \times 24.5 \times 2.5^2 = 15.3$ kN

*Imposed Load*

Imposed load $= 4\text{kPa} = 4 \times 6.5^2 = 169.0\text{kN}$

Ultimate load on floor, $\text{F} = 1.4 \times (259 + 15.3) + 1.6 \times 169$
$= 654 \text{ kN} = {}^{654}/_{6.5^2} = 15.5 \text{ kN/m}^2$

Width of Column Strip and Middle Strip

The drop dimension is greater than one-third of panel dimension, therefore, the column strip is taken as the width of the drop panel = 2.5 m

The width of the middle strip $= 6.5 - 2.5 = 4$ m Figure 6.9

Moment at Interior Support and Span

*Centre of Interior Span*

Sagging moment $= 0.063Fl = 0.063 \times 654 \times 6.5 = 267.8$ kNm Table 6.4

As middle strip is wider than half of the panel dimension, the moment would be increased in proportion,

$0.45 \times \frac{4}{6.5/2} = 0.55$ Table 6.10

Middle strip sagging moment $= 0.55 \times 267.8 = 147.3$ kNm

Column strip sagging moment $= 0.45 \times 267.8 = 120.5$ kNm

*Interior Support*

Hogging moment $= -0.063Fl = -0.063 \times 654 \times 6.5 = -265$ kNm Table 6.4

This moment is reduced by

$0.15Fh_c = 0.15 \times 654 \times 1.4 = 137.3$ kNm Clause 6.1.5.2(g)

Hence the hogging moment $= 265 - 137.3 = 127.7$ kNm

Middle strip hogging moment $= 0.25 \times \frac{4}{6.5/2} \times 127.7 = 39.3$ kNm Table 6.10

Column strip hogging moment $= 127.7 - 39.3 = 88.4$ kNm

Design for the Reinforcement for the Strips

At the drop, the effective depth of the inner layer $= 350 - 25 - 12 - 6 = 307$ mm
At the slab, the effective depth of the inner layer $= 250 - 25 - 12 - 6 = 207$ mm

For middle strip (sagging moment),

$$\mathrm{K} = \frac{M}{f_{cu}bd^2} = \frac{147.3 \times 10^6}{30(4000)(207)^2} = 0.029$$

$$\mathrm{z} = \left(0.5 + \sqrt{0.25 - K/0.9}\right)d = 0.97d > 0.95d = 196 \ mm$$

$$A_s = \frac{M}{0.87f_yz} = \frac{147.3\times10^6}{0.87(500)(196)} = 1728 \ \mathrm{mm}^2$$

Provide 16T12, $A_s$ = 1810mm$^2$

For column strip (sagging moment),

$$\mathrm{K} = \frac{M}{f_{cu}bd^2} = \frac{120.5 \times 10^6}{30(2500)(207)^2} = 0.037$$

$$\mathrm{z} = \left(0.5 + \sqrt{0.25 - K/0.9}\right)d = 0.96d > 0.95d = 196 \ mm$$

$$A_s = \frac{M}{0.87f_yz} = \frac{120.5\times10^6}{0.87(500)(196)} = 1413 \ \mathrm{mm}^2$$

Provide 13T12, $A_s$ = 1469 mm$^2$

For middle strip (hogging moment),

$$\mathrm{K} = \frac{M}{f_{cu}bd^2} = \frac{39.3 \times 10^6}{30(4000)(207)^2} = 0.008$$

$$\mathrm{z} = \left(0.5 + \sqrt{0.25 - K/0.9}\right)d = 0.99d > 0.95d = 196 \;\; mm$$

$$A_s = \frac{M}{0.87f_yz} = 461 \;\; \mathrm{mm}^2$$

Provide 13T12, $A_s$ = 1469 mm$^2$

For column strip (hogging moment),

$$\mathrm{K} = \frac{M}{f_{cu}bd^2} = \frac{88.4 \times 10^6}{30(2500)(307)^2} = 0.013$$

$$\mathrm{z} = \left(0.5 + \sqrt{0.25 - K/0.9}\right)d = 0.99d > 0.95d = 291 \;\; mm$$

$$A_s = \frac{M}{0.87f_yz} = 699 \;\; \mathrm{mm}^2$$

Provide 13T12, $A_s$ = 1469 mm$^2$

<u>Check for Punching Shear</u>

*At the column head*

Perimeter of column head, $u = 3.14 \times 1400 = 4398 \;\; \mathrm{mm}$

Shear force, $V = \left(6.5^2 - {1.4^2 \times 3.14}/{4}\right) \times 15.5 = 630 \;\; \mathrm{kN}$

*V* is increased by 15% for the effect of moment transfer Figure 6.12

$$v = 1.15 \times {630 \times 10^3}/{(4398 \times 307)} = 0.54\,\mathrm{N/mm^2} < 0.8\sqrt{30}$$

$$= 4.38\,\mathrm{N/mm^2}$$

Clause 6.1.5.6(d)

At 1.5*d* from column face

Perimeter $= 4(1400 + 2 \times 1.5 \times 307) = 9284 \;\; \mathrm{mm}$

Shear force, $V = (6.5^2 - (1.4 + 2 \times 1.5 \times 0.307)^2) \times 15.5 = 571 \;\; \mathrm{kN}$

Clause 6.1.5.7(f)

$$v = {571 \times 10^3}/{(9284 \times 307)} = 0.2\,\mathrm{N/mm^2} < v_c$$

## 6.2 Members axially loaded with or without flexure

### 6.2.1 Columns

*Short and slender columns:*
Columns are classified into *short columns* and *slender columns*, which have to be dealt with in different ways. A column may be considered as short when both $l_{ex}/h$ and $l_{ey}/b$ are less than 15 if the column is braced or 10 if the column is unbraced. Otherwise, it should be considered as slender. The effective height $l_e$ of a column in a given plane is equal to $\beta l_0$ where $l_0$ is the clear height of the column. If both ends of the column are restrained (i.e. are connected to other members), the clear height $l_0$ of the column is the clear height between the end restraints. In such case, the clear height $l_0$ should not exceed 60$b$. If one end is unrestrained (i.e. is a free end not connected to any other members), the clear height of the column is the vertical distance from the clear surface of the restrained end to the free end. In such case, the clear height $l_0$ should not exceed min. (60$b$ or $100b^2/h$). Four types of end conditions are defined. The first three are applicable to both braced and unbraced columns whereas the fourth one (free end condition) is applicable only to unbraced columns. Depending on the combination of end conditions at top and bottom, the value of the factor $\beta$ may be obtained from Table 6.11 for braced columns and from Table 6.12 for unbraced columns. From these tables, it can be seen that for braced columns, the values of $\beta$ are generally smaller than or equal to 1.0 while for unbraced columns, the values of β are generally larger than or equal to 1.2. Hence, the slenderness ratios (i.e. $l_{ex}/h$ and $l_{ey}/b$) of unbraced columns tend to be larger.

Worked Example 6.5: RC column classification

Determine the column in the braced frame as shown in Figure 6.8 as short or slender column. Column size is 600 x 550 mm.

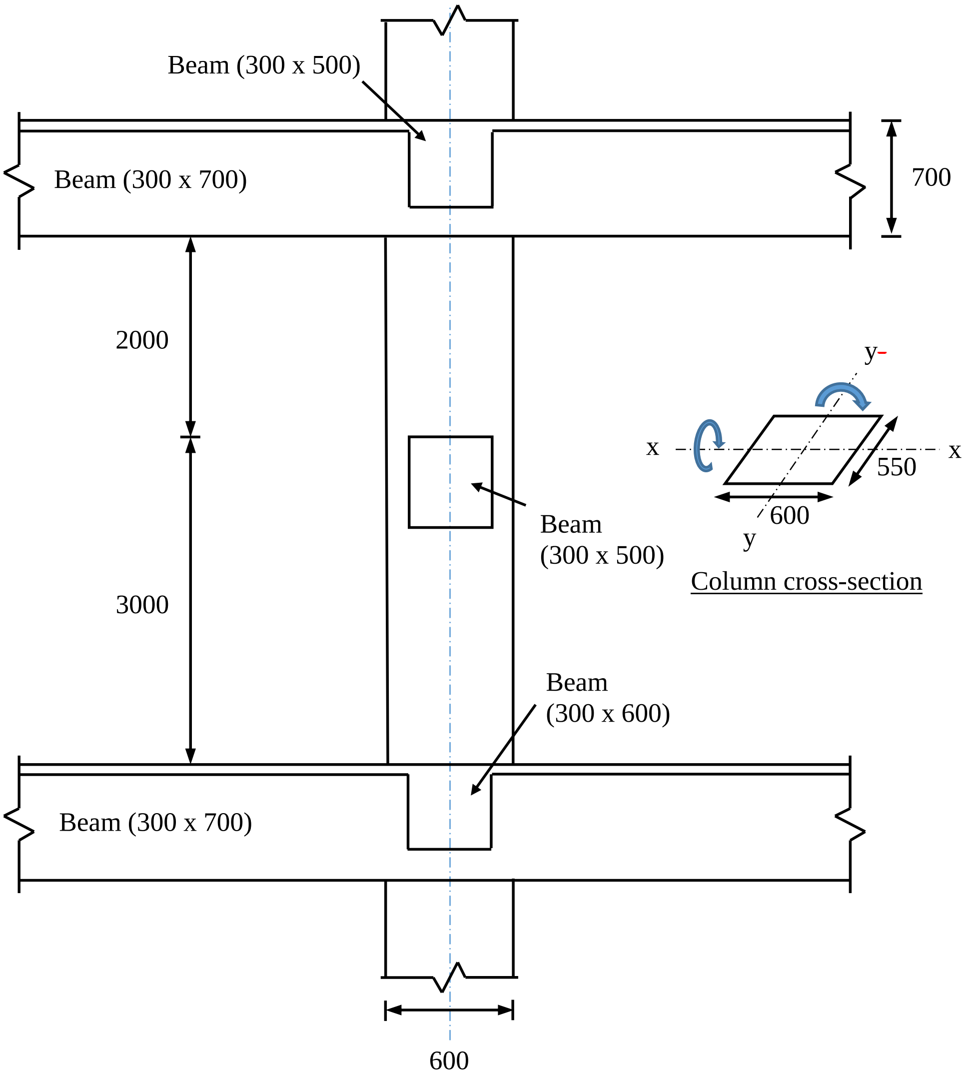



Figure 6.8 Worked example – RC column classification

**About y - axis**

Clear height $l_0 = 5000$ mm

| | Beam depth / column width | Condition |
|---|---|---|
| Top end | 700 / 600 > 1 | 1 |
| Bottom end | 700 / 600 > 1 | 1 |

Table 6.11

Hence, $\beta = 0.75$

Effective height

$$l_{ey} = \beta l_0 = 0.75 \times 5000 = 3750 \text{ mm}$$ Equation 6.46

Effective height to width ratio

$$\frac{l_{ey}}{b} = \frac{3750}{600} = 6.25 < 15$$

The column is considered as short column. Clause 6.2.1.1

**About x - axis**

Segment above the intermediate beam

$$l_0 = 2700 - 500 = 2200 \text{ mm}$$

| | Beam depth / column width | Condition |
|---|---|---|
| Top end | 500 / 550 < 1 | 2 |
| Bottom end | 500 / 550 < 1 | 2 |

Table 6.11

Hence, $\beta = 0.85$

Effective height

$$l_{ex} = \beta l_0 = 0.85 \times 2200 = 1870 \text{ mm}$$ Equation 6.46

Effective height to width ratio

$$\frac{l_{ex}}{b} = \frac{1870}{550} = 3.4 < 15$$

Segment below the intermediate beam

$$l_0 = 2500 \text{ mm}$$

| | Beam depth / column width | Condition |
|---|---|---|
| Top end | 500 / 550 < 1 | 2 |
| Bottom end | 600 / 550 > 1 | 1 |

Table 6.11

Hence, $\beta = 0.80$

Effective height

$$l_{ey} = \beta l_0 = 0.8 \times 2500 = 2000 \text{ mm}$$ Equation 6.46

$$\frac{l_{ey}}{b} = \frac{2000}{550} = 3.64 < 15$$

The column is considered as short column. Clause 6.2.1.1

*Moments and forces in columns:*
The moments and forces in the columns due to the design loads shall be evaluated by analysis of structure as per Chapter 5 of the code. However, if the column being considered is a slender column, the moment due to the design loads should be added with the deflection induced moment (also referred to as additional moment induced by deflection) to obtain the design moment at ULS. Then for any column, regardless of whether it is a short or slender column, the design moment at ULS should not be taken as smaller than the design ultimate axial load multiplied by a minimum eccentricity $e_{\min}$, which may be set equal to the minimum of (0.05 times overall dimension of column in the plane being considered or 20 mm).

*Deflection induced moments at ULS:*
The deflection of a rectangular or circular column at ULS may be taken to be:

$$a_u = \beta_a K h \qquad \text{Equation 6.48}$$

in which:

$$\beta_a = \frac{1}{2000}\left(\frac{l_e}{b}\right)^2 \qquad \text{Equation 6.51}$$

$$K = \frac{N_{uz} - N}{N_{uz} - N_{\text{bal}}} \leq 1 \qquad \text{Equation 6.49}$$

$\beta_a$ is to allow for the slenderness of the column. Its variation with $l_e/b$ has been tabulated in Table 6.13 of the code. On the other hand, $K$ is a reduction factor that corrects the deflection to allow for the influence of axial load on the size of compression zone in the column section (note that $K < 1$ when $N > N_{\text{bal}}$ and $K = 1$ when $N \leq N_{\text{bal}}$). The deflection induces an additional moment given by:

$$M_{\text{add}} = N a_u \qquad \text{Equation 6.52}$$

The distribution of the above additional moment along the height of the column is dependent on whether the column is braced and on the end conditions.

For a braced column, if both ends are not providing rotational restraint, the distribution of additional moment is like a 1/2 cycle sine curve with a maximum value at mid-height. If one end is providing rotational restraint while the other end is not, the distribution of additional moment is like a 3/4 cycle sine curve with a maximum value at approximately 0.4 times column height from the end with no rotational restraint. If both ends are providing rotational restraint, the distribution of additional moment is a full cycle sine curve with a maximum value at mid-height. The distribution of additional moment and the resulting design moment envelopes for different end conditions are shown in Figure 6.9. For an unbraced column, if one end is providing rotational restraint while the other end is not (i.e. a free end), the distribution of additional moment is like a 1/4 cycle sine curve with a maximum value at the end with rotational restraint. If both ends are providing rotational restraint, then the distribution of additional moment is like a 1/2 cycle sine curve with maximum values at the ends. The distribution of additional moment and the resulting design moment envelopes for different end conditions are shown in Figure 6.10.

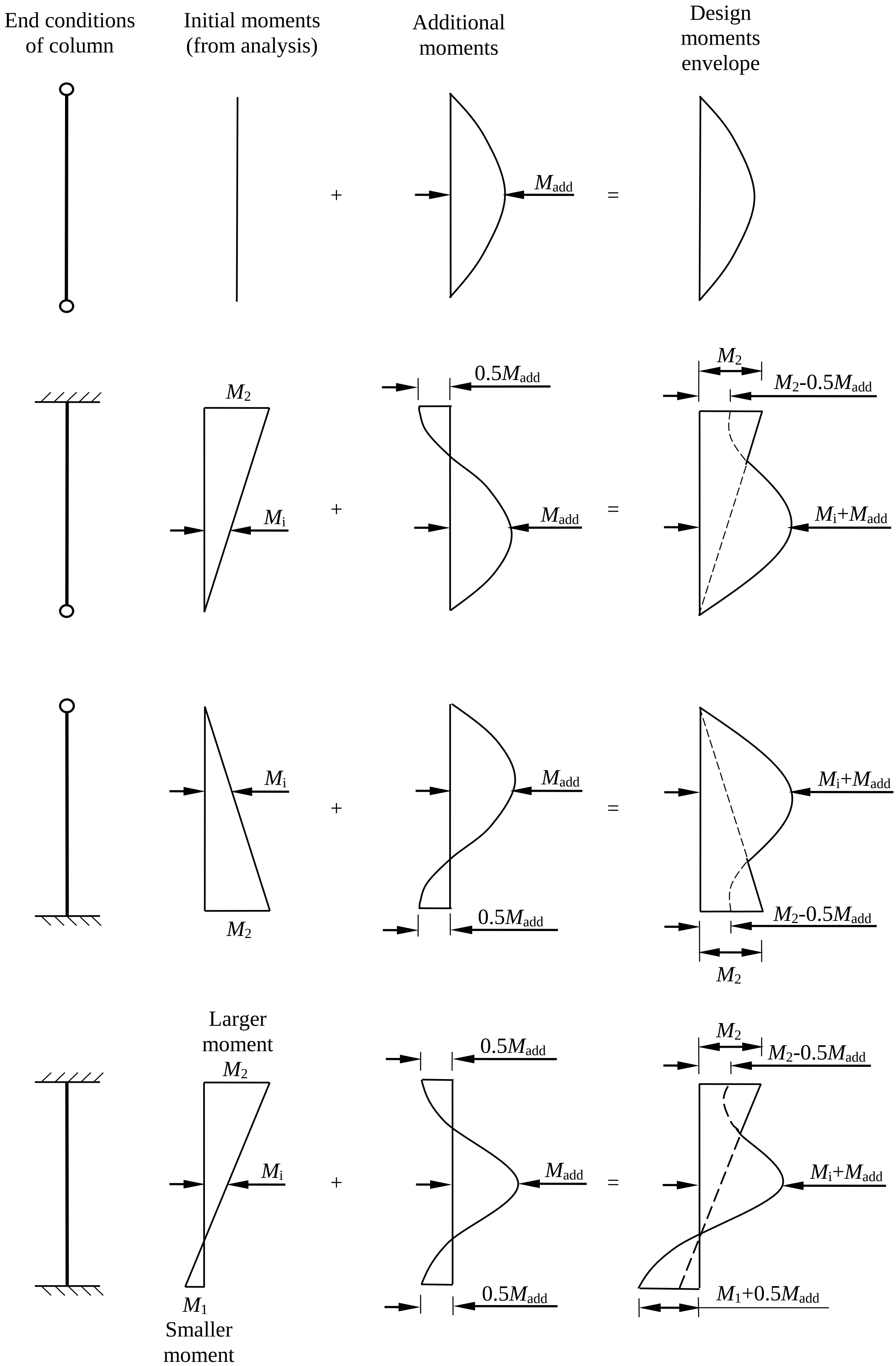



Figure 6.9 Design moment envelopes – braced columns

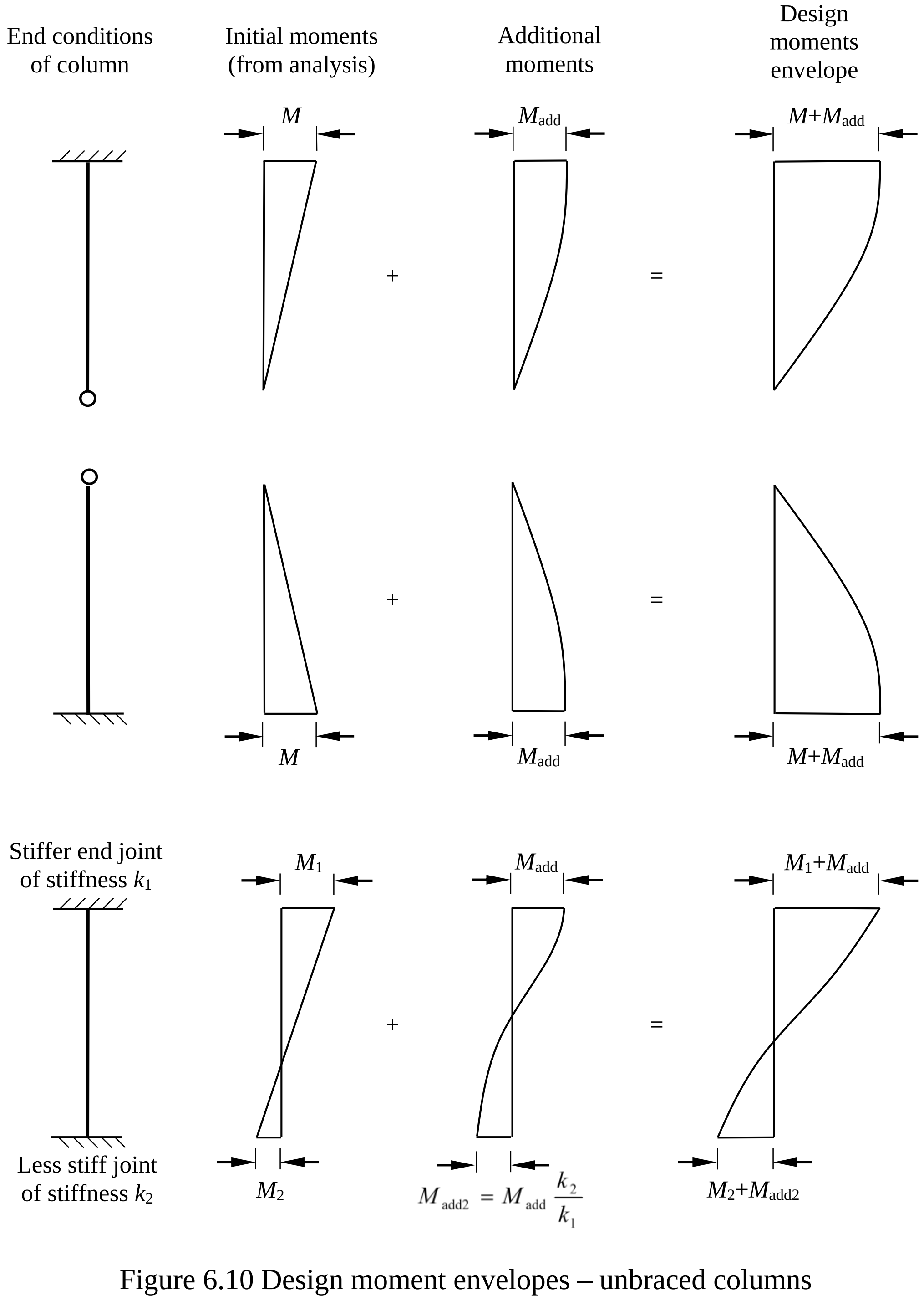



Figure 6.10 Design moment envelopes – unbraced columns

For unbraced columns at the same storey level, which are constrained to deflect sideways by the same amount, an average ultimate deflection $a_{\text{uav}}$ may be applied to all the columns. The average deflection $a_{\text{uav}}$ may be evaluated as $\frac{1}{n}\sum a_u$ in which $n$ is the number of columns.

In theory, all slender columns should be designed for biaxial bending. However, Clause 6.2.1.3(c) allows slender columns bent about a single axis to be designed only for uniaxial bending provided $h/_b < 3$ and $l_e/_h \leq 20$.

*Design of column sections for ULS:*
In the analysis of column sections for ULS, the same assumptions should be made as for the analysis of beam sections. For a short column subjected only to axial load, the ultimate axial load $N$ may be evaluated by:

$$N = 0.4 f_{cu} A_c + 0.75 f_y A_{sc} \qquad \text{Equation 6.5}$$

For a short braced column supporting an approximately symmetrical arrangement of beams, the ultimate axial load *N* may be evaluated by:

$$N = 0.35 f_{cu} A_c + 0.67 f_y A_{sc} \qquad \text{Equation 6.56}$$

For a column to be designed for biaxial bending, Clause 6.2.1.4(d) of the code provides a simple method, which is applicable only if the column section is rectangular and symmetrically reinforced. In the more general case of an arbitrarily shaped column section, such as a circular section, an L-shaped section, a polygonal-shaped section and a section with a hole etc, a more rigorous and generally applicable method will have to be used. Fully computerized design methods for columns subjected to axial load and biaxial bending may be found in the following publications:

Kwan K.H. and Liauw T.C., "Computerized ultimate strength analysis of reinforced concrete sections subjected to axial compression and biaxial bending", Computers and Structures, Vol.21, No.6, 1985, pp1119-1127.

Kwan K.H. and Liauw T.C., "Computer aided design of reinforced concrete members subjected to axial compression and biaxial bending", Structural Engineer, Part B, Vol.63B, No.2, June, 1985, pp34-40.

Liauw T.C. and Kwan K.H., "Computerized design of reinforced concrete columns by load factor method", Journal, Hong Kong Institution of Engineers, February, 1988, pp23-30; also in: Transactions, Hong Kong Institution of Engineers, Vol.1, 1989, pp3-10.

When an axial compressive load applies to a short concrete member, it will be subjected to a uniform strain. If a moment with zero axial load is applied to the same member, the result will be bending about the member's neutral axis such that strain is proportional to the distance from the neutral axis. When the axial load and moment are applied at the same time, the resulting strain will be a combination of two linear diagrams and will itself be linear as shown in Figure 6.11.

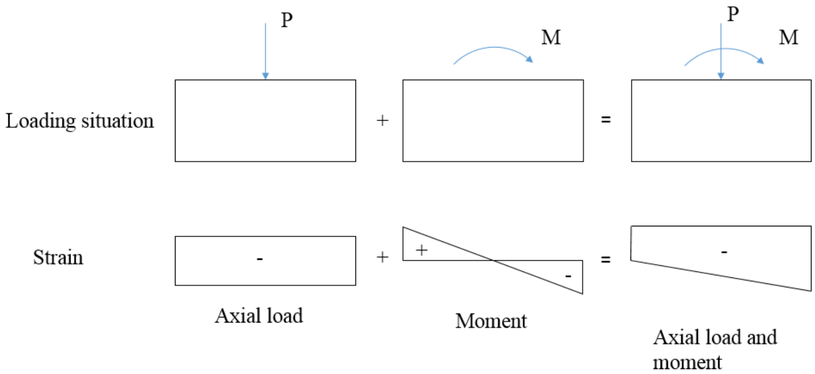



Figure 6.11 Strain distribution under axial load and moment

As the axial load applied to a column is changed, the moment that the column can resist will be changed. The ultimate strain with different moment and axial force combination is shown in Figure 6.12. Assuming the concrete on the compression edge of the column will fail at a strain of

$$f_{cu} \leq 60\text{MPa},\ \ \varepsilon_{cu} = 0.0035 \qquad \text{Figure 3.8}$$

$$f_{cu} > 60\text{MPa},\ \ \varepsilon_{cu} = 0.0035 - 0.00006\sqrt{(f_{cu} - 60)}$$

A strain can be assumed on the far edge of the column, the values of axial load *N* and moment *M* can be computed by static. A series of *N* and *M* values is determined to correspond with an ultimate strain on the compression edge.

If the reinforcement closest to the extreme tension side of the column reaches yield strain, the column is said to be tension controlled; otherwise it is compression controlled. The transition point between these regions is the balance point. The balance section refers to a section whose compression concrete strain reached ultimate strain at the same time as the tensile reinforcement reached its yield strain.

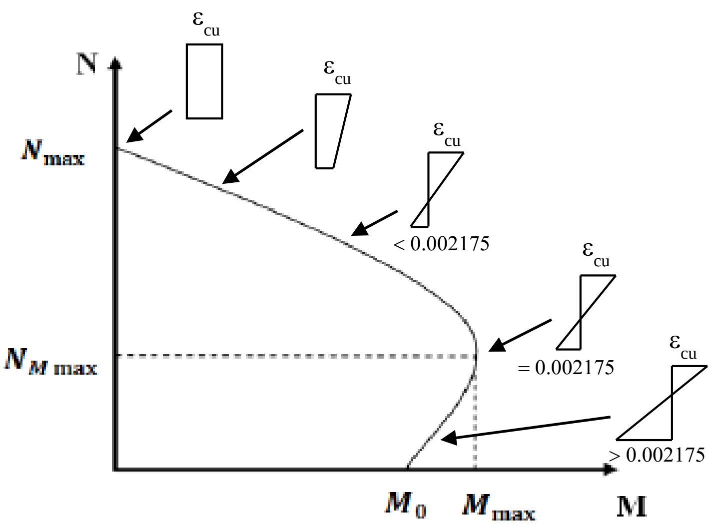



Figure 6.12: *M*-*N* interaction diagram with various ultimate strain

Worked Example 6.6: RC Column Design

A frame as shown in Figure 6.13 of a heavily loaded industrial structure for which the column C1, C2 and C3 are to be designed. The frams at 4m c/c are braced against lateral forces.

Dead load = 5 kPa
Live load = 15 kPa
Characteristic material strength, C50 for concrete
Column size: 400 x 400
Beam size: 300 x 700
All columns are considered as short.

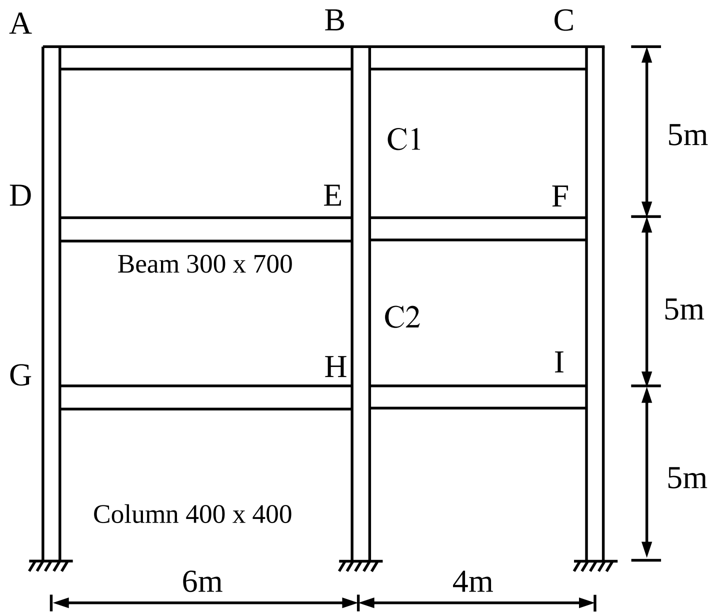



Figure 6.13 Worked example – RC column design

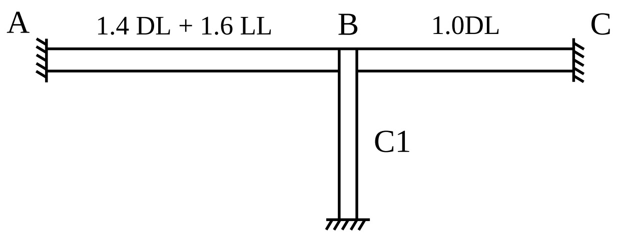



Figure 6.13(a) Sub-frame for third floor

For design of column C1 (top), loading combination is as follows:

$$1.4\text{DL} + 1.6\text{LL} = [1.4(5) + 1.6(15)] \times 4 = 124\ \text{kN/m} \qquad \text{Table 2.1}$$

$$1.0\text{DL} = [1.0(5)] \times 4 = 20 \ \text{kN/m}$$

$$FEM_{AB} = \frac{124(6)^2}{12} = 372 \ \text{kNm}$$

$$FEM_{BC} = \frac{20(4)^2}{12} = 26.7 \ \text{kNm}$$

$$K_{AB} = \frac{1}{2}\frac{bh^3}{12L} = \frac{1}{2}\frac{300(700)^3}{12(6000)} = 7.14 \times 10^5 \ \text{m}^3$$

$$K_{BC} = \frac{1}{2}\frac{bh^3}{12L} = \frac{1}{2}\frac{300(700)^3}{12(4000)} = 1.07 \times 10^6 \ \text{m}^3$$

$$K_{C_1} = \frac{bh^3}{12L} = \frac{400(400)^3}{12(5000)} = 4.27 \times 10^5 \ \text{m}^3$$

Distribution factor for column $= \frac{4.27\times10^5}{7.14\times10^5+1.07\times10^6+4.27\times10^6} = 0.193$

Moment at column C1 (top) $= 0.193(372 - 26.7) = 66.6 \ \text{kNm}$

Design axial force $= 124 \times 3 + 20 \times 2 = 412 \ \text{kN}$

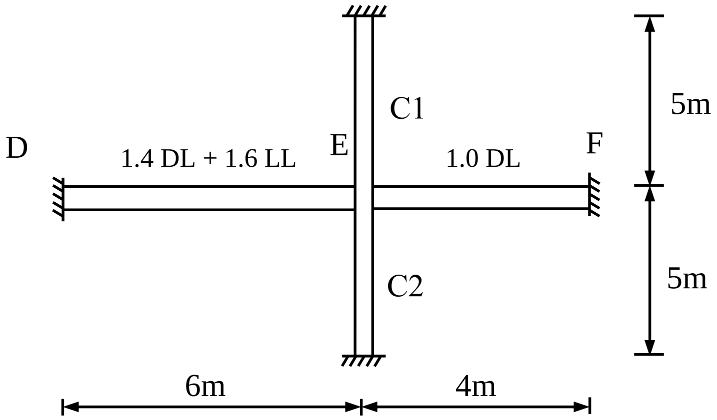



Figure 6.13(b) Sub-frame for second floor

For design of column C1 (bottom) and column C2 (top), loading combination is as follows:

$$1.4\text{DL} + 1.6\text{LL} = [1.4(5) + 1.6(15)] \times 4 = 124\text{kN/m}$$ Table 2.1

$$1.0\text{DL} = [1.0(5)] \times 4 = 20\text{kN/m}$$

$$FEM_{AB} = \frac{124(6)^2}{12} = 372\text{kNm}$$

$$FEM_{BC} = \frac{20(4)^2}{12} = 26.7\text{kNm}$$

$$K_{AB} = \frac{1}{2}\frac{bh^3}{12L} = \frac{1}{2}\frac{300(700)^3}{12(6000)} = 7.14 \times 10^5\text{m}^3$$

$$K_{BC} = \frac{1}{2}\frac{bh^3}{12L} = \frac{1}{2}\frac{300(700)^3}{12(4000)} = 1.07 \times 10^6\text{m}^3$$

$$K_{C_1} = \frac{bh^3}{12L} = \frac{400(400)^3}{12(5000)} = 4.27 \times 10^5 \mathrm{m}^3$$

$$K_{C_2} = \frac{bh^3}{12L} = \frac{400(400)^3}{12(5000)} = 4.27 \times 10^5 \mathrm{m}^3$$

Distribution factor for C1 (bottom) and C2 (top) = 0.162

Maximum moment at C1 (top) and C2 (bottom)

$= 0.162(372 - 26.7) = 55.9 \ \ \mathrm{kNm}$

Design axial force for C1 (bottom)

$= 124 \times 5 + 20(\mathrm{column\ \ self-weight}) = 640 \ \ \mathrm{kN}$

Design axial force for C2 (top)

$= 124 \times 5 + 124 \times 3 + 20 \times 2 + 20(\mathrm{column\ \ self-weight}) = 1052 \ \ \mathrm{kN}$

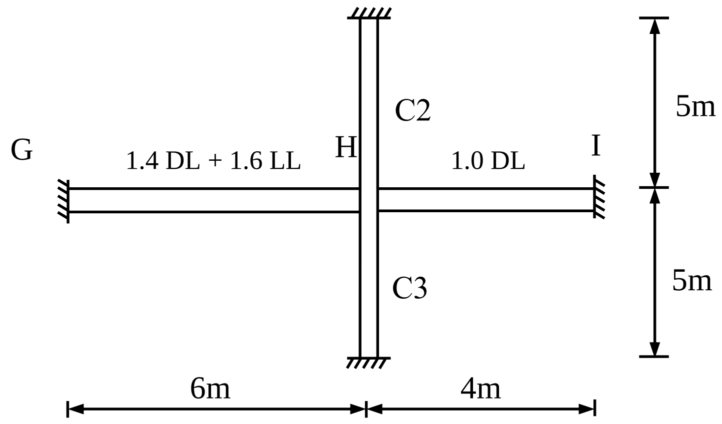



Figure 6.13(c) Sub-frame for first floor

For design of C2 (bottom) and C3 (top), the sub-frame above can be considered. As all dimenions are the same, the results are as follows:

Maximum moment at C2 (bottom) and C3 (top)

$= 0.162(372 - 26.7) = 55.9 \ \ \mathrm{kNm}$

Design axial force for C2 (bottom)

$= 124 \times 5 \times 2 + 20(\mathrm{column\ \ self-weight}) \times 2 = 1280 \ \ \mathrm{kN}$

Design axial force for C2 (top)

$= 124 \times 5 \times 2 + 124 \times 3 + 20 \times 2 + 20(\mathrm{column\ \ self-weight}) \times 2 = 1692 \ \ \mathrm{kN}$

Note: In the code of practice, for design of columns and walls, it requires to consider the following cases (a) maximum axial load combined with coexistent bending moment; (b) minimum axial load combined with coexistent bending moment; (c) Maximum bending moment combined with coexistent axial load; (d) any other coexistent combination which will be more critical to above cases. The example above illustrates one of the cases and more analyses should be performed to identify the critical design moment and axial force for column reinforcement design.

### 6.2.2 Walls

*General:*

The overall stability of a multi-storey building should not, in any direction, depend on the out-of-plane rigidity of the walls alone. Lateral supports to walls should each be designed to resist, in the out-of-plane direction, the sum of the design ultimate horizontal load acting on the wall (due to the applied loads) and a horizontal load equal to 2.5% of the design ultimate vertical load (for preventing lateral buckling). From the design point of view, walls are classified into reinforced walls and plain walls. A reinforced wall is a concrete wall containing at least the minimum quantities of reinforcement specified in Clauses 9.6.1 to 9.6.4 of the code. A plain wall is a concrete wall containing no reinforcement or insufficient reinforcement to satisfy the minimum quantities of reinforcement specified in Clauses 9.6.1 to 9.6.4 of the code; for a plain wall, any reinforcement is ignored when considering the strength of the wall.

*Design of reinforced walls:*

For a reinforced wall constructed monolithically with the adjacent construction, the effective height $l_e$ should be assessed as though the wall is a column subjected to bending at right angle to the wall (i.e. out-of-plane bending). For a reinforced concrete wall supporting the beams and slabs which are simply supported on the wall, the effective height $l_e$ should be assessed as though the wall is a plain wall. Except for short braced walls loaded almost symmetrically, the eccentricity $e_{\min}$ of the axial load in the direction at right angle to the wall (i.e. in the out-of-plane direction) should be taken as not less than 0.05 times the thickness of wall or 20 mm, whichever is lesser (similar to that for columns).

Clause 6.2.2.2(d) of the code suggests that when a horizontal force is resisted by the in-plane action of several walls, the proportion allocated to each wall should be in proportion to its relative stiffness. However, the term “relative stiffness” is difficult to define and for many years has been loosely interpreted by different engineers in totally different ways. In fact, from the theory of structure point of view, this postulation that the horizontal shear should be apportioned to each wall in proportion to its relative stiffness is wrong. The distribution of the horizontal load to the walls is dependent on many factors including spatial distribution of the walls, torsional stiffness of the building and coupling between the walls, as illustrated in Figure 6.14, and wall-frame interaction, as explained before.

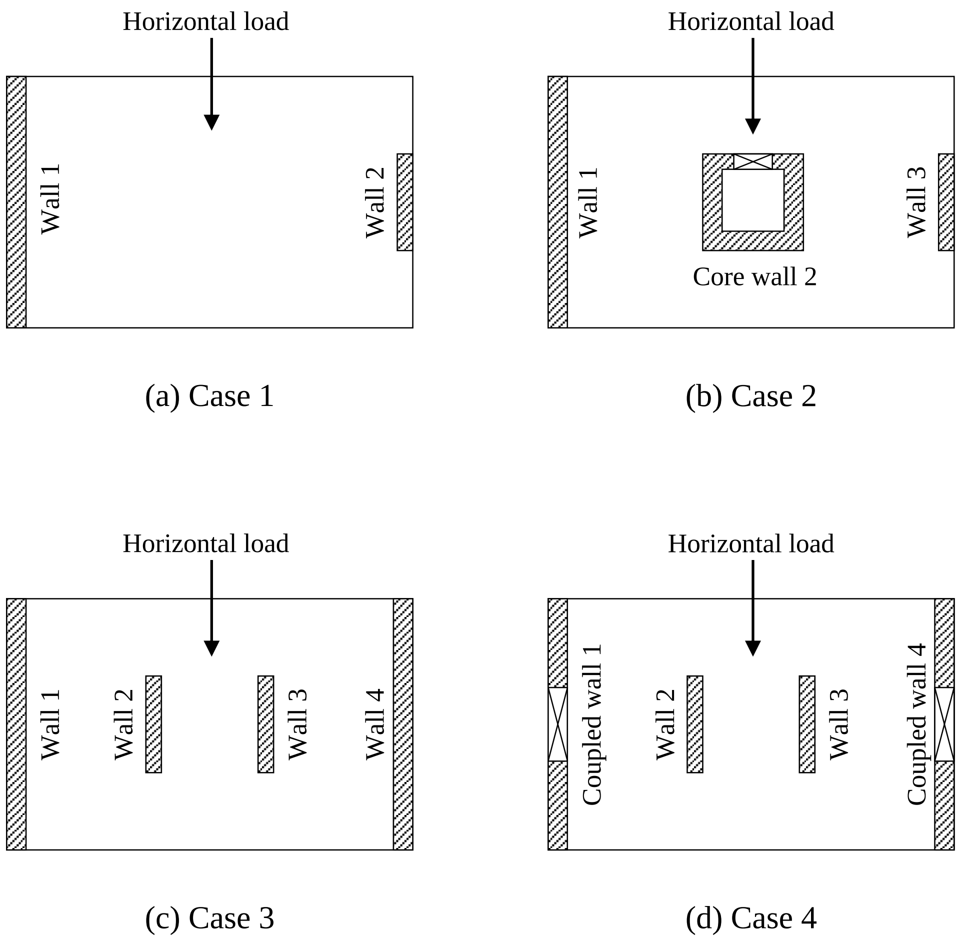



Figure 6.14: Distribution of horizontal load to walls

Four different cases are shown in Figure 6.14. In Case 1, a horizontal load is applied to the geometric centre of a building with two shear walls of different lateral stiffness each at one side of the building. Although the stiffness of the two shear walls are different, because of the necessity to satisfy the torsional equilibrium condition, the loads distributed to the two shear walls are the same, i.e. each of the two shear walls is allocated one half of the applied horizontal load. In Case 2, a horizontal load is applied to the centre of a building with a core wall and two shear walls of different stiffness. In this particular case, because of the high torsional stiffness of the core wall, the building would deflect laterally with little twisting. As a result, the horizontal load is distributed to the core wall and the shear walls in proportion to their lateral stiffness. In Case 3, a horizontal load is applied to the centre of a building with a symmetrical arrangement of shear walls. Because of symmetry, the building would deflect laterally with no twisting. As a result, the horizontal load is distributed to the walls in proportion to their lateral stiffness. In Case 4, a horizontal load is applied to the centre of a building with four shear walls, two of which are solid walls while the other two have window openings and are in effect coupled shear walls. The problem in this particular case is that it is very difficult to determine the lateral stiffness of the two coupled shear walls, which do not deflect like solid cantilever walls. Because of the difference in the deflection modes of the coupled shear walls and the solid shear walls, the horizontal load is not distributed to the walls in proportion to their lateral stiffness.

In view of the afore-mentioned situations that may arise in many multi-storey buildings, it is not really appropriate to suggest that the horizontal shear should be apportioned to each wall in proportion to its relative stiffness. The distribution of horizontal loads to the shear/core walls should be analysed by a three-dimensional finite element analysis program with the shear walls modelled as plane stress or shell elements taking into account the spatial distribution of the shear walls, torsional rigidity of the core walls, coupling effect between shear walls and wall-frame interaction.

Regarding the design of the walls, reinforced walls are classified into stocky reinforced walls (i.e. reinforced walls with ${l_e}/{h} < 15$ if braced or with ${l_e}/{h} < 10$ if unbraced) and slender reinforced walls (i.e. reinforced walls other than stocky reinforced walls). They are analogous to short and slender columns. The design of stocky reinforced walls and slender reinforced walls are governed by Clauses 6.2.2.2(f) and 6.2.2.2(g), respectively.

*Design of plain walls:*
The author does not recommend the incorporation of any plain walls in multi-storey buildings, especially tall buildings, the main reason being that in multi-storey buildings, basically all the walls are key elements that are indispensable and need to be designed to withstand a design ultimate load of 34N/m$^2$ in any direction including the out-of-plane direction. Such a high-intensity design ultimate load would demand more than nominal reinforcement to be provided in the walls. Hence, it is inappropriate to design any walls in multi-storey buildings as plain walls. At the most, in a multi-storey building, plain walls should be used for non-structural partition walls only.

## 6.3 Torsion and combined effects

It is said in the code that in "normal" slab-and-beam or framed construction, specific considerations of torsional effects are not usually necessary. There is, however, the question of what is normal and what is not normal. The decision on whether to consider torsional effects should be based on whether the torsional stiffness of the member being considered would have significant effect on the load distribution of the structure. If the torsional stiffness of the member could have certain effects on the load distribution, the torsional stiffness of the member should be taken into account in the analysis; otherwise the torsional stiffness of the member may be ignored. For example, when the member being considered does not lie in a vertical plane (i.e. the member is curved horizontally or has a horizontal bend along its length), the torsional stiffness is likely to have some effects on the load distribution and therefore should be taken into account in the analysis. As another example, when a horizontal member is connected at an angle to another horizontal member in such a way that the flexural rotation of a member would become the torsional rotation of the other member, the torsional stiffness of the member subjected to torsional rotation should be considered in the analysis. Throughout the years, the author has seen quite a number of torsionally cracked beams in Hong Kong, which were most probably due to the belief of some design engineers that "specific considerations of torsional effects are not usually necessary". The author is of the opinion that design engineers should better have a second thought and always satisfy themselves that the torsional effects are really negligible and are not going to cause torsional cracking before neglecting any torsional effects.

In any case, when the torsional resistance of any member is relied on to carry loading, the torsional stiffness, torsional reinforcement and torsional cracking of the member should all be carefully considered. However, the author does not recommend the reliance on the torsional resistance of any member to carry loading, unless there is no other alternative.

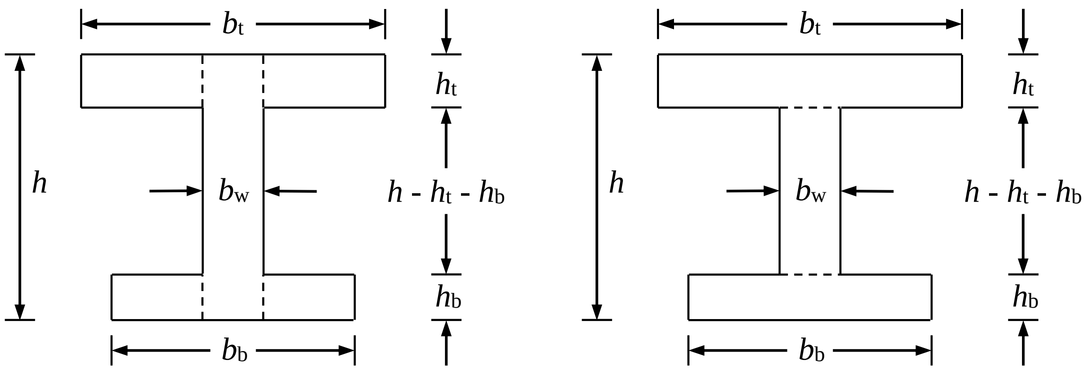
The main difficulty in the evaluation of the torsional constant *C* of a concrete section is that it is dependent on the extent of cracking of the concrete section. In the case of a concrete section subjected to flexure, it can be assumed that all the concrete within the tension zone as demarcated by the position of the neutral axis has cracked, but in the case of a concrete section subjected to torsion, no similar strategy can be applied. To avoid such difficulty, it is assumed in the code that the torsional constant *C* may be taken as half the St. Venant value (the value evaluated for the whole section assuming there is no cracking). The torsional constant *C* is given by:

$$C = \frac{1}{2}\left(\beta {h_{\mathrm{min}}}^{3} h_{\mathrm{max}}\right) \qquad \text{Equation 6.64}$$

in which $\beta$ is a coefficient depending on the shape of the section,
$h_{\mathrm{min}}$ is the smaller dimension of the section,
$h_{\mathrm{max}}$ is the larger dimension of the section, and
the bracketed term $\left(\beta {h_{\mathrm{min}}}^{3} h_{\mathrm{max}}\right)$ is the St. Venant torsional constant.

The above should clarify the second paragraph in Clause 6.3.2 of the code (where the term "St. Venant torsional constant" should be replaced by the term "torsional constant" and Equation 6.64 therein should be amended accordingly). Further explanations can be found in the following book:
A.H. Allen, Reinforced Concrete Design to BS8110 – Simply Explained, E. & F.N. Spon Ltd, London and New York, 1988, 239pp.

The torsional stiffness of a non-rectangular section may be obtained by dividing the section into a number of rectangles and summing up the torsional stiffness of these rectangles. The division of the section should be arranged so as to maximize the calculated torsional stiffness, as illustrated in Figure 6.15 for the case of an I-section.




$C_1 = [\beta_1 b_w^3 h + \beta_2 h_t^3 (b_t - b_w) + \beta_3 h_b^3 (b_b - b_w)]/2$ $\qquad$ $C_2 = [\beta_4 h_t^3 b_t + \beta_5 h_b^3 b_b + \beta_6 b_w^3 (h - h_t - h_b)]/2$

$C$ = maximum of ($C_1$ or $C_2$)

Figure 6.15: Calculation of torsional constant for an I-section

To evaluate the torsional shear stress in the section, the torsion carried by each rectangular component needs to be calculated first. It may be assumed that the torsional moment carried by each rectangular component is proportional to its torsional stiffness, or in other words, given by the following equation:

$$T_i = T \times \frac{{h_{\min}}^3 h_{\max}}{\Sigma({h_{\min}}^3 h_{\max})} \qquad \text{Equation 6.66}$$

in which is $T_i$ the torsional moment acting on a rectangular component and $T$ is the torsion acting on the whole section. Having obtained the torsional moment acting on each rectangular component, the torsional shear stress $v_t$ in any rectangular component may be determined as:

$$v_t = T_i \times \frac{2}{{h_{\min}}^2 \left(h_{\max} - {h_{\min}}/_{3}\right)} \qquad \text{Equation 6.65}$$

The above equation is derived by assuming a plastic shear stress distribution circulating round the rectangle.

If the combined shear stress due to shear and torsion exceed the value of $v_{t\min}$ in Table 6.17, torsional reinforcement should be provided but in no case should the combined shear stress due to shear and torsion exceed the value of $v_{tu}$ in Table 6.17. The values of $v_{t\min}$ and $v_{tu}$ given in Table 6.17 are based on those given in Table 2.3 of BS8110: Part 2: 1985, but have been modified for increased shear strength when concrete of strength grade higher than C40 are used.

The stipulations for provision of torsional reinforcement in the code are the same as those in BS8110: Part 2: 1985 except that the material safety factor of the torsional reinforcement has been set at 1.15 instead of 1.05 in the British Standard. Basically, the torsional reinforcement should consist of rectangular closed links together with longitudinal reinforcement and is additional to any requirements for shear or bending.

## 6.4 Design for robustness against disproportionate collapse

Clause 2.2.2.3 of the code has demanded, for the purpose of achieving structural integrity, that the structural elements should be effectively tied together using the following types of ties:

- peripheral ties;
- internal ties;
- horizontal ties to columns and walls;
- vertical ties to vertical load-bearing elements.

The ties should be provided with tension reinforcement effectively anchored to the structural elements that are being tied together. Reinforcement provided for other purposes may be regarded as forming part of, or the whole of, these ties.

*Peripheral ties:*
At each floor and roof level, an effectively continuous peripheral tie capable of resisting a tensile force of $F_t$ should be provided within 1.2m of the edge of the building or within the perimeter wall. $F_t$ (in kN) is the lesser of $(20 + 4n_0)$ or 60, where $n_0$ is the number of storeys.

*Internal ties:*
At each floor and roof level, internal ties in two directions, approximately at right angles, should be provided. They should be effectively continuous throughout their lengths and should be anchored to the peripheral ties at each end. They may, in whole or in part, be spread evenly in the slabs or may be grouped at beams, walls or other appropriate positions, but at spacings not greater than $1.5l_r$ where $l_r$ is the greater of the distances between the centres of the columns, frames or walls supporting any two adjacent floor spans in the direction of the tie under consideration. In each direction, the ties should be capable of resisting a tensile force (in kN/m width) equal to the greater of:

$$\frac{(G_k+Q_k)}{7.5} \times \frac{l_r}{5} \times F_t \text{ or } F_t \qquad \text{Equation 6.69}$$

where $(G_k + Q_k)$ is the sum of the characteristic dead and imposed floor loads.

*Horizontal ties to columns and walls:*
Each external column and every metre length of each external wall carrying vertical loads should be anchored or tied horizontally into the structure at each floor and roof level with a horizontal tie capable of resisting a tensile force equal to the greater of:

- $2.0F_t$ or $\frac{l_s}{2.5}F_t$ if less, where $l_s$ is the floor to ceiling height (in m); or
- 3% of the total design ultimate vertical load carried by the column or walls.

For a corner column, horizontal ties should be provided in two directions approximately at right angles. Each horizontal tie should be capable of resisting the above tensile force.

*Vertical ties to vertical load-bearing elements:*
Each column and wall carrying vertical loads should be tied continuously from the lowest to the highest level. The tie should be capable of resisting a tensile force equal to the maximum design ultimate dead and imposed load received by the column or wall from any one storey.

There are only two other international codes giving explicit design requirements for the provision of ties in concrete buildings. They are BS8110: Part 1: 1997 and Eurocode 2. The requirements of these two codes are not the same. Generally, the tie forces given in BS8110 are based on the number of storeys while those of Eurocode 2 are based on span length. Hence, direct comparison between these two codes is rather difficult. However, the much smaller tie force requirements for internal ties and horizontal ties to columns in Eurocode 2 compared to that stipulated in BS8110 does not appear to have been justified. It is therefore considered more appropriate to follow BS8110. The tie requirements given in the new code are the same as those outlined in BS8110.

Other than the provision of ties, the necessity to provide bridging elements should also be considered. According to Clause 6.4.2 of the code, a design check should be carried out to evaluate, for each storey in turn, the scenario of losing any one of the vertical load-bearing elements. The design requirement is that the loss of any vertical load-bearing element would not cause the collapse of a significant part of the structure. The vertical load-bearing elements to be considered lost in turn are the columns and the walls. In the case of a column, the whole column would have to be considered lost. In the case of a wall, only the length of the wall between adjacent lateral supports or between a lateral support and a free edge needs to be considered lost.

According to Clauses 2.2.2.3 and 6.4.1.7 of the code, all vertical load-bearing elements should be provided with vertical ties. If, somehow, it is inappropriate or impossible to provide effective vertical ties to all the vertical load-bearing elements, then the vertical load-bearing elements not provided with vertical ties should be considered to be removed in turn and the vertical loads normally carried by each such element in question should be transferred through *bridging elements* to the adjacent vertical load-bearing elements. Catenary action may be employed to transfer the vertical loads but allowance should be made for the horizontal reactions necessary for equilibrium.

## 6.5 Corbels and nibs

A corbel is a short cantilever projection satisfying the following conditions:

- ${a_v}/{d} < 1$
- the depth of the projection at outer edge of contact area of supported load is not less than one-half of the depth at the root of the projection

Corbels may be designed using the strut-and-tie model, with the concrete and reinforcement assumed to act as struts and ties, respectively. All reinforcement should be properly anchored. At the front face, the reinforcement should be anchored either by welding to a transverse bar or by bending back to form a loop. Shear reinforcement should be provided in the form of horizontal links distributed in the upper two-thirds of the effective depth at the root of the corbel.

Where a continuous nib is less than 300 mm deep, it should be designed as a short cantilever slab. The tension reinforcement should project from the supporting member across the top of the nib to a point as near to the front face of the nib as the cover requirement permits. It should be anchored at the front face either by welding to a transverse bar or by bending through 180° to form a loop.

<u>Worked example 6.7: Design of corbel</u>

As shown in Figure 6.16, the corbel has a width $b = 300$ mm and length $l = 400$ mm. Design the reinforcement for the corbel to support an ultimate load on $N = 800$ kN at a distance $a_v = 220$ mm from the face of the column. The main tension reinforcement has an effective depth of $d = 500$ mm. The characteristic material strengths are $f_{cu} = 45\ \text{N/mm}^2$, $\varepsilon_{cu} = 0.0035$, $f_y = 500\ \text{N/mm}^2$.

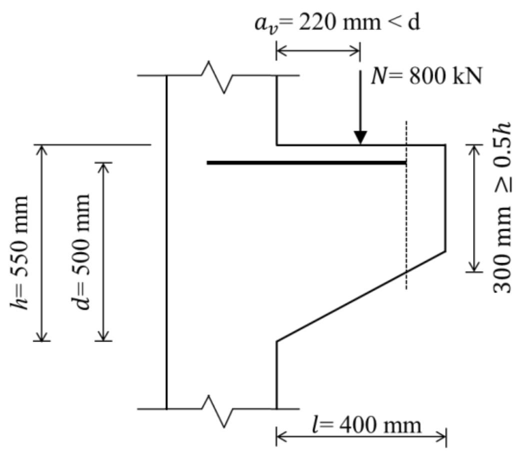



Figure 6.16 Worked example – Corbel design

<u>Design for the tension steel</u> (Cl. 6.5.2):

RC corbel is defined as the disturbed region, also known as D-region or local region, where stress and strain distributions are disturbed or discontinuous. Strut-and-Tie Model (STM) is one of the analytical methods commonly adopted for corbels (Cl. 6.5.2.1). In this model, the applied loading $N$ is resisted by a truss that is formed by a concrete compressive strut and a steel tensile tie, as shown in Figure 6.17 (a).

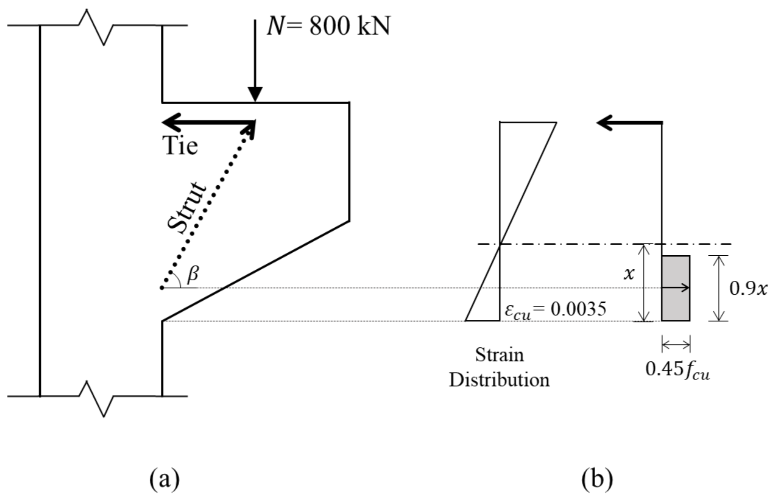



Figure 6.17 (a) Strut-and-tie Model for a Corbel; (b) Strain distribution and stress block of concrete at corbel support

The lateral force of concrete strut in corbel can be provided by the concrete compressive block, as shown in Figure 2 (b), and by Cl. 6.1.2.4, the capacity of the stress block can be simplified as $0.45f_{cu} \times b \times 0.9x = 0.405f_{cu}bx$ for $f_{cu} \leq 45\text{N/mm}^2$ . By trigonometry, the concrete strut compression can be expressed as $F_c = 0.405f_{cu}bx \cos \beta$ and the applied load $N = F_c \sin \beta$. Also, $\tan \beta$ can be solved as $(\mathrm{d} - 0.45\mathrm{x})/a_v$ by geometry, therefore $\cos \beta = a_v/\sqrt{a_v^2 + (d - 0.45x)^2}$ and $\sin \beta = (d - 0.45x)/\sqrt{a_v^2 + (d - 0.45x)^2}$. By substituting the trigonometric functions into the force equilibrium equation:

$$N = F_c \sin\beta = 0.405f_{cu}bx \cos \beta\sin \beta = 0.405f_{cu}bx \frac{a_v(d - 0.45x)}{a_v^2 + (d - 0.45x)^2}$$

Re-arrange this equation as a quadratic equation about $x$:

$$(0.2025N + 0.18225f_{cu}ba_v)x^2 - 0.9d(N + 0.45f_{cu}ba_v)x + N({a_v}^2 + d^2) = 0,$$

Solve the equation and $x = 326.07$ mm.

By assuming that plain remains plain and linear strain distribution, the strain at steel level $\varepsilon_s$:

$\varepsilon_s = \frac{d-x}{x}\varepsilon_{cu} = \frac{500-326.07}{326.07} \times 0.0035 = 0.00187 < 0.002175 = 0.87f_y/E_s$, so the

tension steel has not yielded yet.

The stress in the top tension steel is $f_s = E_s \times \varepsilon_s = 200 \times 10^3 \times 0.00187 = 373.37\ \text{N/mm}^2$

The force in the top steel is $T = \mathrm{N}\cot\beta = \frac{Na_v}{d-0.45x} = \frac{800\times220}{500-0.45\times326.07} = 498\ \ \mathrm{kN} > 0.5\times 800 = 400\ \ \mathrm{kN}$

Required tension steel area $A_s = \frac{T}{f_s} = \frac{498000}{373.37} = 1334\ \ \mathrm{mm}^2$
5T20 is provided as tension steel and the steel area is $A_s = 5\times314 = 1570\ \ \mathrm{mm}^2$.

<u>Design for the shear reinforcement</u> (Cl. 6.5.2):

By Table 6.3, Note 2: $v_c$ (without enhancement) $= 0.79\times(\frac{100A_s}{bd})^{\frac{1}{3}}(\frac{400}{d})^{\frac{1}{4}}\frac{1}{\gamma_m} = 0.79\times(\frac{100\times1334}{300\times500})^{\frac{1}{3}}(\frac{400}{500})^{\frac{1}{4}}\frac{1}{1.25} = 0.739\ \ \mathrm{N/mm^2}$, (Cl. 6.1.2.5 (d)).
With enhancement, it becomes $v_c = \frac{2d}{a_v}0.739 = 3.361\ \ \mathrm{N/mm^2}$.
Shear stress $= \frac{800\times10^3}{500\times300} = 5.33\ \ \mathrm{N/mm^2} > \ \ 3.361\ \ \mathrm{N/mm^2}$.

So the shear reinforcement is provided as:

$\frac{A_{sv}}{s_v} = \frac{b(v-v_c)}{0.87f_y} = \frac{300(5.33-3.361)}{0.87\times500} = 1.36\ \ \mathrm{mm}$,

Total shear area provided $= \frac{A_{sv}}{s_v}d = 1.36\times500 = 680\ \ \mathrm{mm}^2$

Horizontal shear check: shear reinforcement area > half area of tension steel, where

$\frac{A_{sv}}{s_v}d \geq \frac{1}{2}A_s = \frac{1}{2}\times1570 = 785\ \ \mathrm{mm}^2$.

So take the required shear reinforcement area as $785\ \ \mathrm{mm}^2$,

4T12 is provided as shear reinforcement and the steel area is

$A_{sv} = 4\times113\times2 = \ \ 904\ \ \mathrm{mm}^2$.

The reinforcement detail of the corbel is as shown in Figure 6.18

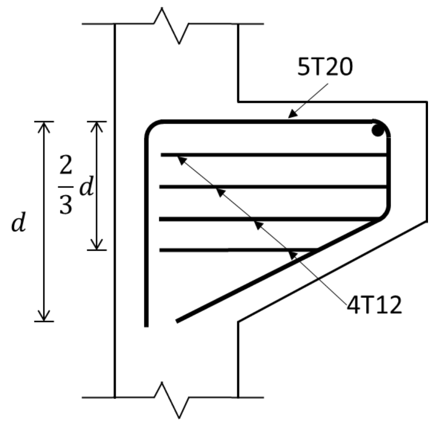



Figure 6.18 Reinforcement Details

## 6.6 Staircases

The provisions given in the new code for the design of staircases are the same as those in BS8110: Part 1: 1997. Generally, staircases should be designed in accordance with the requirements for beams and slabs given in Clauses 6.1.2 and 6.1.3 of the code. The specific issues related to the design of staircases are the depth of section and permissible span/depth ratio. For a staircase, the depth of section should be taken as the minimum thickness perpendicular to the soffit of the staircase. Provided the stair flight occupies at least 60% of the span, the permissible span/depth ratio calculated in accordance with Clause 7.3.4 (this clause is for beams and slabs) may be increased by 15%.

## 6.7 Foundations

For the design of pad footings and pile caps, the "rigid footing or cap" assumption may be adopted for the determination of bearing stresses or pile loads (commonly known as rigid footing or cap design). If the flexibility of the footing or cap is to be considered for the purpose of reducing the bearing stresses or pile loads at the corners of the footing or cap (commonly known as flexible footing or cap design), a rigorous and accurate elastic analysis method, such as the finite element method, should be used.

In the finite element method, the mathematical model for simulation of raft footings and pile caps are popularly formed by plate elements, i.e. a raft footing or a pile cap is modelled as an assembly of plate elements. The model is obviously more realistic than the rigid cap analysis as it takes the flexibility (or rigidity) of the footing and pile cap into account in the analysis and therefore termed flexible footing (or cap) analysis. The analytical results comprises moment fields (bending moments and twisting moment as discussed in the foregoing sections and out-of-plane shear for design.

Nevertheless, it should be noted that in taking the bending stiffness of the plate element in analysis, only the out-of-plane bending and shear are considered under the assumption of linear stress and strain distribution through the depth of the structure. The assumption may not be valid in case of the footing or pile cap is having a thickness comparable to its lateral dimensions especially for spans over piles or other rigid supports. Under such circumstances, the footing or pile cap may behave like a "deep beam" and performing tie-and-strut actions within the structure. So there is a "strut-and-tie" approach by locating the probable direct stress load path within the structure which is in the form of "strut-and-tie" action.

The "strut-and-tie" model can only be formed easily for simple structures. For complicated structures involving irregular shape of footing or pile cap and irregular piling layout, the strut and tie layouts can be of many form. Also as the structure becomes highly statically indeterminate, the assignment of structural sizes to the ties and struts is difficult.

With the advancement of development of the finite element method and the computer hardware, it becomes possible to simulate the footing or pile cap structure as assemblies of solid elements.

The modelling by solid elements is the most advanced and accurate model that can capture the true structure behaviour of the footing or pile cap. However, design will be more difficult as the outputs involves 3-dimensional stress field comprising three direct stresses and 3 shear stresses. There are approaches by Foster et al (2003) and Law et al (2007) for reinforced concrete design to the 3-D stress fields. Nevertheless, it should also be noted if the model is first order elastic, there may be locations at mid-depths of the structure having high tension developed and require reinforcements whereas in the conventional design with the simulation as plate or even beam, reinforcements are often placed near element top and bottom surfaces with concrete cracking assumed beyond the neutral axis. So to avoid placing reinforcements near mid-depths of the structure, elasto-plastic modelling may be adopted with locations where no reinforcements are to be placed with no (or low) tensile strengths.

*Design of pad footings:*
For the design of an isolated footing as a beam or plate bending structure, the critical section may be taken as that at the face of the column or wall supported. While calculating the design ultimate moment, it should be noted that no redistribution of moments is allowed. If the width of the section is relatively large, the tension reinforcement for resisting the ultimate moment should be distributed with two-thirds of the total required concentrating within a zone near the centre of the column supported; otherwise the tension reinforcement should be uniformly distributed, as stipulated in Clause 6.7.2.2 of the code. While designing for shear strength, it should be noted that two shear conditions need to be considered: (1) shear along a vertical section extending across the full width of the footing; and (2) punching shear around concentrated loads.

*Design of pile caps:*
The bending design of a pile cap may be carried out using the bending theory if the pile cap is modelled as a beam or plate structure. For the shear design of a pile cap, the critical sections should be assumed to be located at 20% of the diameter of the pile inside the face of the pile. The shear span $a_v$ should be taken as the distance from the face of the column to the critical section for the shear being considered. Where due consideration has been given to the shear distribution across the section, shear enhancement (see Clause 6.1.2.5) may be applied to the strip of the pile cap of width equal to 3.0 times the pile diameter, centred at each pile. In consideration of the shear distribution across the section, the shear force may be averaged over a width, which should not extend beyond one effective depth on either side from the pile centre, or as limited by the actual dimension of the pile cap. If the shear stress is smaller than the enhanced shear strength, no shear reinforcement is required. Apart from checking the shear along the critical sections, a check should be made to ensure that the punching shear stress at the perimeter of the column is not exceeding min. of ($0.8\sqrt{f_{cu}}$ or 7.0 N/mm$^2$). In addition, if the spacing of the piles is greater than 3.0 times the pile diameter,

punching shear of the pile cap due to the concentrated pile loads should also be checked as per the same requirements for flat slabs (see Clause 6.1.5.7).

Worked Example 6.8: Design of a pile cap:

A four-pile group supports a 1.0 m square column carrying an ultimate axial load of 15,000kN. The piles are 0.9 m in diameter and are spaced at 3.0 m centres. Concrete of grade C35 and high-yield steel reinforcement are to be used. A sketch of the pile cap layout is shown in Figure 6.19.

Load on each pile $= {}^{15000}/_{4} = 3750\ \text{kN}$

Bending moment at face of column $= 2 \times 3750 \times (1.5 - 0.5) = 7500\ \text{kNm}$

$$K = \frac{M}{bd^2 f_{cu}} = \frac{7500\times10^6}{5000\times1200^2\times35} = 0.030 \qquad \text{Equation 6.7}$$

$$z/d = 0.5 + \sqrt{0.25 - \frac{K}{0.9}} = 0.966$$

Take $z = 0.95d = 0.95 \times 1200 = 1140\ \text{mm}$

$$A_s = \frac{M}{0.87 f_y z} = \frac{7500\times10^6}{0.87\times500\times1140} = 15123\ \text{mm}^2$$

Use 34 ϕ25 bars; $A_s$ provided = 16690 mm$^2$ > 15123 mm$^2$, O.K.

Shear force at critical section = 2 × 3750 = 7500 kN

This shear force is to be distributed over a width extending one effective depth on either side of each pile centre or as limited by the actual dimension of the pile cap.

Effective width = 2 × (1200 + 1000) = 4400 mm

Shear stress $= \frac{7500\times10^3}{4400\times1200} = 1.42\ \text{N/mm}^2$

Shear enhancement may only be applied to the strip of pile cap within 1.5 × pile diameter at each side of a pile.

Since 1.5 × pile diameter = 1.5 × 900 = 1350 mm, the whole effective width is within the strip where shear enhancement may be applied.

Shear strength $v_c = 0.45\ \text{N/mm}^2$

Shear span $a_v$= 1500 – 500 – 0.3 × 900 = 730 mm

Effective depth $d$ = 1200 mm

$$\frac{2d}{a_v} = \frac{2\times1200}{730} = 3.29$$

Enhanced shear strength = 3.29 × 0.45 = 1.48 N/mm$^2$

Since the shear stress is smaller than the enhanced shear strength, no shear reinforcement is required.

Punching shear checking is also to be carried out (omitted herein for brevity).

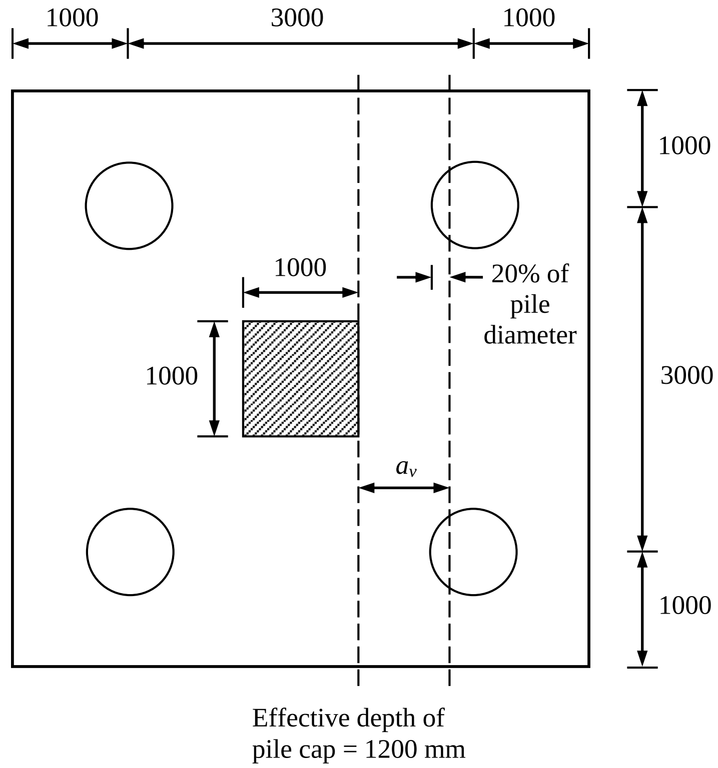



Figure 6.19: Worked example - design of a pile cap

## 6.8 Beam-column joints

The code states that beam-column joints should be designed to satisfy the following criteria:

- at serviceability limit state, a beam-column joint shall perform at least as well as the members that it joins; and
- at ultimate limit state, a beam-column joint shall have a design strength sufficient to resist the most adverse load combinations sustained by the adjoining members.

To be more specific, the basic requirements of a beam-column joint are:

- at serviceability limit state, the beam-column joint should not crack seriously and should not show any signs of spalling; and
- at ultimate limit state, the beam-column joint should be able to maintain its structural integrity so as to allow the full strength potentials of the adjoining members to be developed.

At the vertical interface between the beam and the column, stress concentration occurs and vertical cracks often appear at the tension side of the interface, as shown in Figure 6.18. If the tension reinforcement of the beam has not been properly anchored into the joint, the tension reinforcement tends to be pulled out of the joint and this could result in the formation of fairly wide cracks at the beam-column interface (the crack width is actually the same as the bond-slip of the tension reinforcement). Moreover, if the tension reinforcement is bent into the column in such a way that the bearing stresses induced at the inside of the bend are only resisted by a thin concrete cover, the concrete cover could spall off, as depicted in Figure 6.20. Hence, proper detailing of the reinforcement going into the joint is crucial for the high performance at serviceability limit state of a beam-column joint. This is however easier said than done because there is often the problem that the joint does not have sufficient space to accommodate the required anchorage length of the reinforcing bars coming in from the beams.

Apart from cracking and spalling near the beam-column interface, the joint itself might also fail due to diagonal crushing or splitting. To avoid such kinds of failure, it is necessary to:

- limit the joint shear stress as per Equation 6.71 in Clause 6.8.1.3 of the code,
- provide horizontal joint shear reinforcement as per Equation 6.72 in Clause 6.8.1.5 of the code,
- provide vertical joint shear reinforcement as per Equation 6.73 in Clause 6.8.1.6 of the code, and
- provide confining reinforcement to the length of column forming the beam-column joint as per Clause 6.8.1.7 of the code.

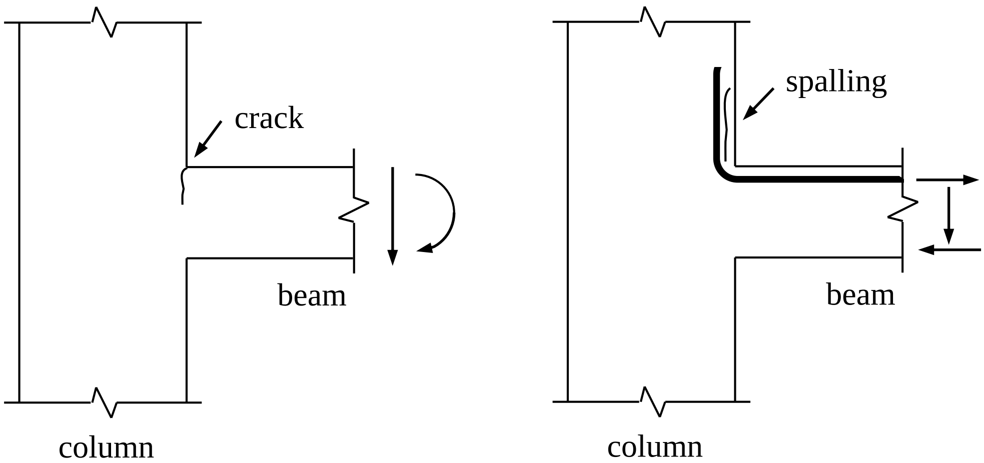



Figure 6.20 Cracking and spalling at beam-column interface

The usual practice in the past of not putting in any joint shear or joint confining reinforcement (commonly known as empty joint design) is no longer considered satisfactory. However, putting all the reinforcement required by the code together, the joint could become quite congested with reinforcing bars. Fixing of the beam reinforcement, the column reinforcement and the joint shear and confining reinforcement at the joint would demand higher skill than before and render placing and compaction of the concrete in the joint much more difficult. Design engineers should be aware of such practical difficulties and should pay particular attention to the various site supervision problems that may arise.

However, all this trouble is necessary for the beam-column joints to maintain their structural integrity at the ultimate limit state.

The provisions in Section 6.8 of the code for the design of beam-column joints are largely based on the New Zealand Standard NZS3101: Part 1: 1995. Part 2 of this standard (NZS3101: Part 2: 1995), which is a commentary handbook, has provided some background information and explanations of the rationale behind the various design requirements for beam-column joints. However, NZS3101 is not easy to follow partly because of the complicated behaviour of beam-column joints and partly because there are many differences between New Zealand and Hong Kong in the way the design loads and strengths are specified. For easier reference and understanding, a brief introduction to the fundamental theory of beam-column joints is presented in the following.

*Joint forces:*
There are two types of beam-column joints, interior beam-column joints and exterior beam-column joints, as shown in Figure 6.21. An interior beam-column joint is a joint that is connected to an internal column and two beams at its two sides in the same vertical plane of the joint. An exterior beam-column joint is a joint that is connected to an external column and one beams at one side in the same vertical plane of the joint.

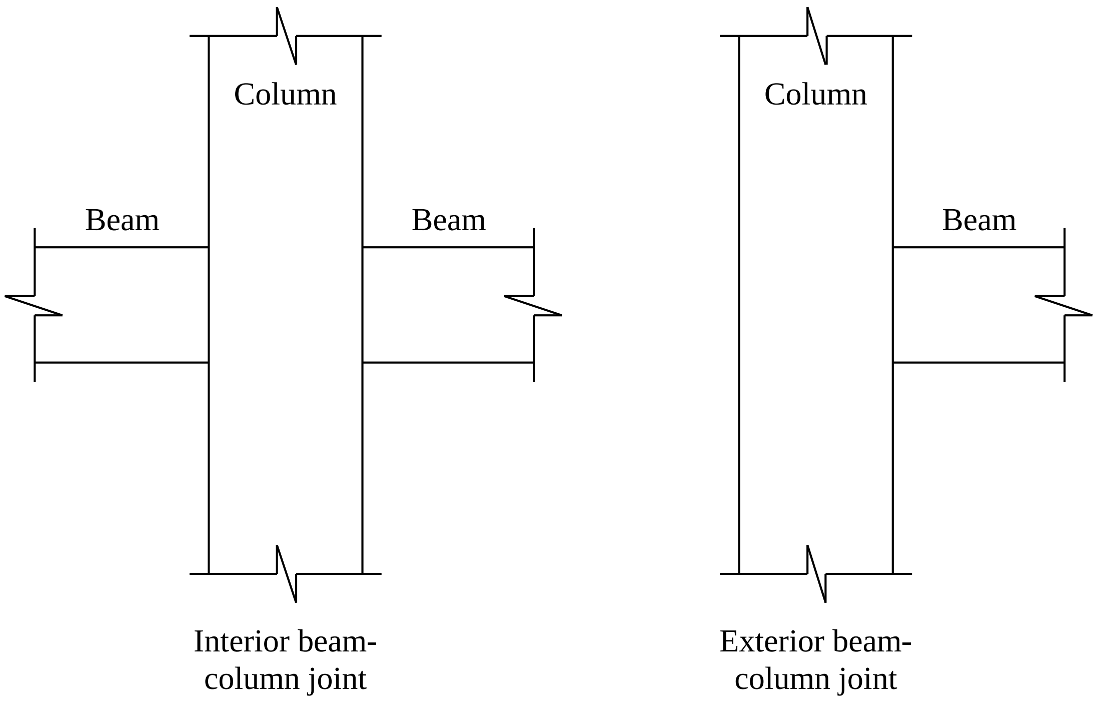



Figure 6.21 Interior and exterior beam-column joints

For a three-dimensional beam-column joint with beams framing into the joint in two different vertical planes (i.e. not all the beams are in the same vertical plane), the forces acting on the beam-column joint need only be considered in each vertical plane at a time. In other words, such a three-dimensional beam-column joint has to be considered twice, each time as a two-dimensional beam-column joint in a vertical plane containing the beams in the same vertical plane. NZS3101 states that the forces in each vertical plane may be considered independently, but in actual fact, the horizontal joint shear force in one vertical plane could affect the joint shear reinforcement design in the other vertical plane through the factor $C_j$ (see Clauses 11.3.5 and 11.3.6 of NZS3101: Part 1 and Clauses 6.8.1.5 and 6.8.1.6 of the code).

The forces acting on a beam-column joint in each vertical plane are:

- axial force $N_1$, horizontal shear force $V_{c1}$ and bending moment $M_{c1}$ from the column above the joint;
- axial force $N_2$, horizontal shear force $V_{c2}$ and bending moment $M_{c2}$ from the column below the joint;
- vertical shear force $V_{b1}$ and bending moment $M_{b1}$ from the beam at one side of the joint; and
- vertical shear force $V_{b2}$ and bending moment $M_{b2}$ from the beam at the other side of the joint, if any.

*Horizontal and vertical joint shear forces:*
There are two possible scenarios:
(a) Gravity load: the two beams at the two sides of the joint are both subjected to hogging moments; and
(b) Lateral load: one beam at one side of the joint is subjected to hogging moment while the other beam at the other side of the joint is subjected to sagging moment.

Scenario(a): Figure 6.22 shows joint loads acting on the free body of the joint with beam moments $M_{b1}$ and $M_{b2}$ acting on opposite faces, in the opposing sense. The beam moments are generally unequal, with their difference equilibrated by the sum of the column moments $M_{c1}$ and $M_{c2}$.

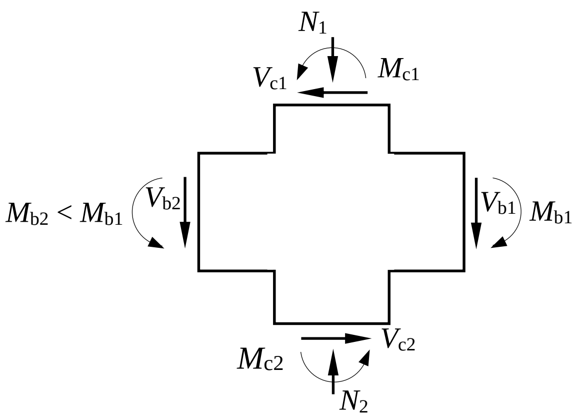



Figure 6.22 Joint loads resulting from gravity loads

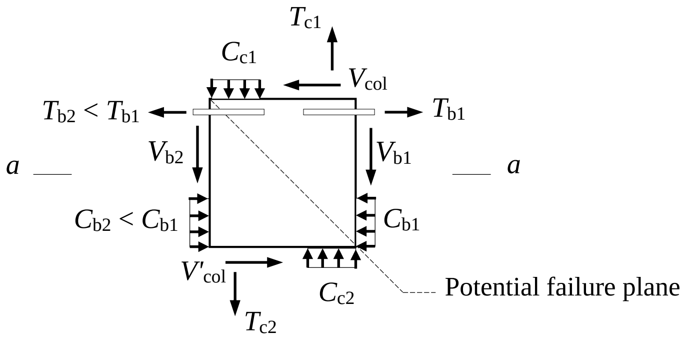



Figure 6.23 Joint forces resulting from gravity loads

In Figure 6.23, the horizontal design shear force of joint (<u>from gravity loads</u>) is given by

$$V_{jh} = T_{b1} - T_{b2} - V_{col}$$

At the centre of the beam-column joint, there is also a vertical joint shear force. The vertical joint shear force $V_{jv}$ may be derived from similar considerations. For simplicity, the vertical joint shear force may be approximated as follows:

$$V_{jv} = \left({h_b}/{h_c}\right) V_{jh}$$

Where $h_b$ is the beam depth, and $h_c$ is the column depth in the direction of the horizontal shear considered.

Scenario (b): Figure 6.24 shows joint loads acting on the free body of the joint with beam moments $M_{b1}$ and $M_{b2}$ acting on opposite faces, in the same sense. The beam moments are generally unequal, with their sum equilibrated by the sum of the column moments $M_{c1}$ and $M_{c2}$.

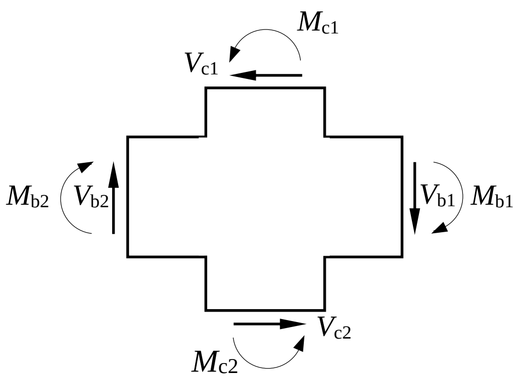



Figure 6.24 Joint loads resulting from lateral loads

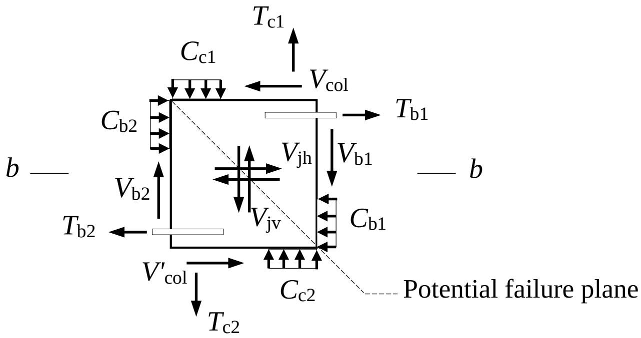



Figure 6.25 Joint forces resulting from lateral loads

the horizontal design shear force of joint (from lateral loads) is given by

$$V_{jh} = T_{b1} + C_{b2} - V_{col}$$

From equilibrium, the compression force at the joint face $C_{b2} = T_{b2}$; hence

$$V_{jh} = T_{b1} + T_{b2} - V_{col}$$

At the centre of the beam-column joint, there is also a vertical joint shear force. The vertical joint shear force $V_{jv}$ may be derived from similar considerations. For simplicity, the vertical joint shear force may be approximated as follows:

$$V_{jv} = \left({h_b}/{h_c}\right) V_{jh}$$

Where $h_b$ is the beam depth, and $h_c$ is the column depth in the direction of the horizontal shear considered.

*Horizontal and vertical joint shear reinforcement:*

The horizontal and vertical joint shear forces are to be resisted by a truss mechanism comprising of a diagonal compression strut in the concrete and the horizontal and vertical joint shear reinforcement. To resist the horizontal joint shear force, account may be taken of the contribution of the diagonal compression strut to shear resistance and the increase in concrete shear strength due to the axial compression on the column. To resist the vertical joint shear force, account may be taken of the contribution of the diagonal compression strut to shear resistance and the counterbalancing effect of the axial compression on the column. Taking into account the contribution of the diagonal compression strut and the effects of the axial compression on the column, the code has provided the following formulas for evaluating the area of horizontal joint shear reinforcement $A_{jh}$ and the area of vertical joint shear reinforcement $A_{jv}$:

$$A_{jh} = \frac{V_{jh}^*}{0.87 f_{yh}} \left(0.5 - \frac{C_j N^*}{0.8 A_g f_{cu}}\right) \quad \text{Equation 6.72}$$

$$A_{jv} = \frac{0.4V_{jv}^{*} - C_jN^{*}}{0.87f_{yv}} = \frac{0.4\left(^{h_b}/_{h_c}\right)V_{jv}^{*} - C_jN^{*}}{0.87f_{yv}} \qquad \text{Equation 6.73}$$

in which $V_{jh}^{*}$, $V_{jv}^{*}$ and $N^{*}$ are the values of $A_{jh}$, $V_{jv}$ and $N$ at the ultimate limit state and $C_j$ is the factor for allocating the beneficial effect of the axial compression to the two horizontal directions of the joint (or the two vertical planes containing the beams). If the horizontal joint shear forces in the two horizontal directions of the joint are $V_{jx}$ and $V_{jy}$, then $C_j$ would be given by:

$$C_j = \frac{V_{jh}}{V_{jx} + V_{jy}} \qquad \text{Equation 6.72}$$

*Maximum allowable joint shear stress:*

To safeguard the core concrete (i.e. the diagonal compression strut in the concrete) from crushing failure, an upper limit needs to be set to the joint shear stress. The new code has imposed the following limit:

$$\frac{V_{jh}^{*}}{b_jh_c} \leq 0.2f_{cu}$$

in which $b_j$ is the effective joint width and $h_c$ is the depth of the column in the direction of the horizontal shear being considered. In case the above limit is exceeded, regardless of how much shear reinforcement is to be added to the joint, the joint size (mainly the column size) will have to be enlarged until the above joint shear stress is kept within limit.

*Confinement:*
In addition to the horizontal and vertical joint shear reinforcement, horizontal transverse confining reinforcement also has to be provided. The horizontal transverse confining reinforcement in beam-column joints shall not be less than that required by Clause 9.5.2, with the exception of joints connecting beams at all four column faces in which case the transverse confining reinforcement may be reduced to one-half of that required by Clause 9.5.2. In no case shall the link spacing of the transverse confining reinforcement exceed 10 times the diameter of the smallest column bar or 200 mm, whichever is the smaller.

For additional background information of beam column joint, the following paper may be referred: Wong, H.F. and Kuang, J.S. “On the design of RC beam-column joints to Hong Kong Code 2004”, HKIE Transactions, Vol. 16, No. 3, 2009, pp 48-55.

Two worked examples are given to show the design and detailing of

(1) an interior joint without vertical joint shear reinforcement, and
(2) an exterior joint with vertical joint shear reinforcement

Worked example 6.9: Design of Interior beam-column joints

Figure 6.26 shows the interior joint of a reinforced concrete frame building, with beam and column dimensions and reinforcing steel as indicated. The frame is designed to carry gravity and normal wind loads, and the storey height is 3.7 m. The minimum design axial load of the column is 2500 kN. Material strengths are $f_{cu}$ = 35 N/mm$^2$ and $f_y$ = 500 N/mm$^2$. Design the joint reinforcement.

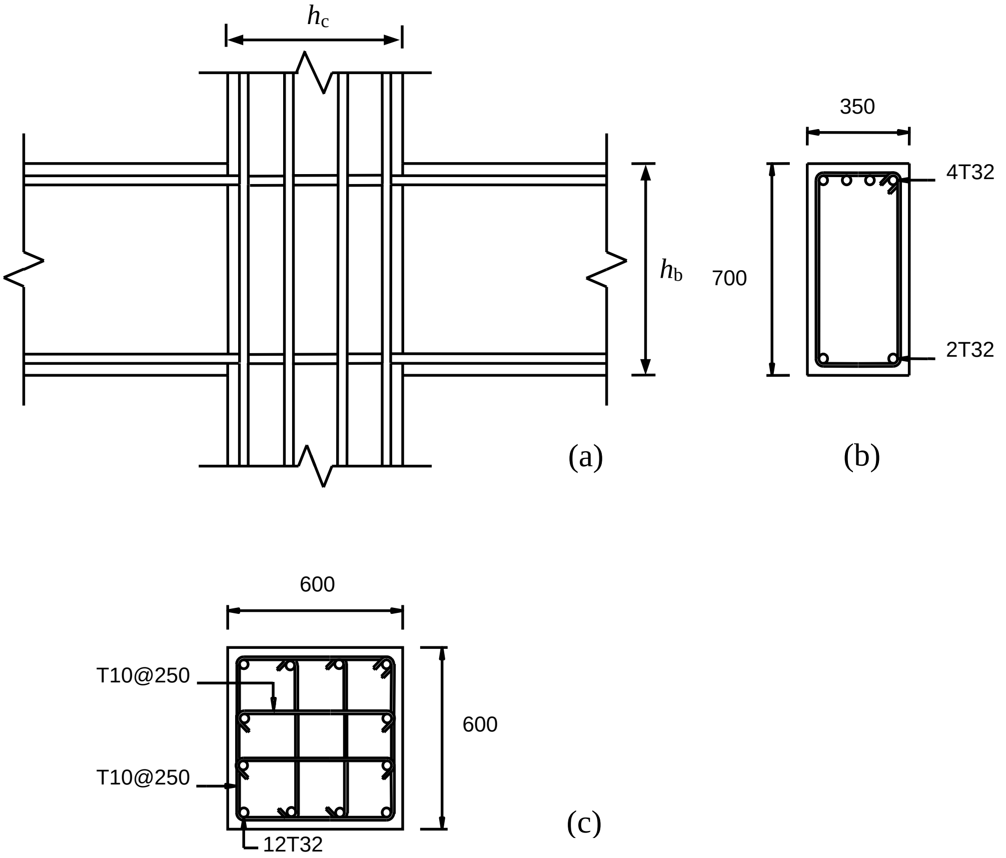



Figure 6.26 Interior beam-column joint. (a) Beam-column connection; (b) beam cross-section; (c) column cross-section of non-critical zones

There are two load cases which correspond to the joint horizontal shear forces. Since it is a lateral load resisting frame, assuming maximum joint shear arises when a hogging plastic hinge and a sagging plastic hinge form in beams. Therefore, the horizontal joint shear force is as follows:

The maximum tension forces in beam top and bottom reinforcing bars are

$$T_{b1} = A_{s1} f_y = 3217 \times 500 \times 10^{-3} = 1608.5 \text{ kN}$$
$$T_{b2} = A_{s2} f_y = 1608 \times 500 \times 10^{-3} = 804 \text{ kN}$$

From equilibrium, $A_s f_y = 0.67 f_{cu} b(0.9x)$; then $x = {}^{A_s f_y}\big/_{0.603 f_{cu} b}$. Hence for the beam top and bottom reinforcement,

$$x_{b1} = \frac{A_{s1}f_y}{0.603f_{cu}b} = \frac{1608.5\times10^3}{0.603\times35\times350} = 218\ mm$$

$$x_{b2} = \frac{A_{s2}f_y}{0.603f_{cu}b} = \frac{804\times10^3}{0.603\times35\times350} = 109\ mm$$

The corresponding design moments applied at the join faces are

$$M_{u1} = A_{s1}f_y\left(d - \frac{0.9x_{b1}}{2}\right) = 1608.5\times10^3\left(630 - \frac{0.9\times218}{2}\right) = 855.6\text{kNm}$$

$$M_{u2} = A_{s2}f_y\left(d - \frac{0.9x_{b2}}{2}\right) = 804\times10^3\left(630 - \frac{0.9\times109}{2}\right) = 467.1\text{kNm}$$

Column shears corresponding to the joint moments shown in Figure 6.27 are calculated by

$$V_{col} = \frac{M_{u1}+M_{u2}}{l_c} = \frac{(855.6+467.1)\times10^6}{3700} = 357.5kN$$

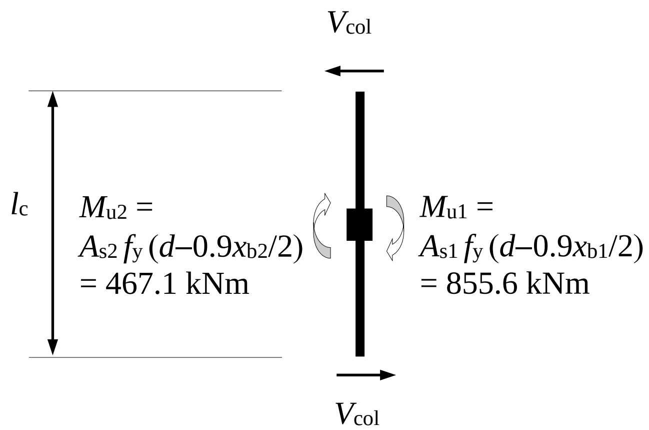



Figure 6.27 Column shear calculated from free body of column between points of contraflexure

$$V_{jh} = T_{b1} + T_{b2} - V_{col} = 1608.5 + 804 - 357.5 = 2055\text{kN}$$

Check the horizontal shear stress:

$$\frac{V_{jh}}{b_jh_c} = \frac{2055\times10^3}{600\times600} = 5.71\,\text{N/mm}^2 < 0.2f_{cu} = 7\,\text{N/mm}^2 \qquad \text{Equation 6.71}$$

This is satisfactory.

For a two-dimensional joint, $C_j = 1.0$; hence the area of horizontal joint shear reinforcement

$$A_{jh} = \frac{V_{jh}}{0.87 f_{yh}}\left(0.5 - \frac{C_j N^*}{0.8 A_g f_{cu}}\right) \qquad \text{Equation 6.72}$$

$$A_{jh} = \frac{2055\times10^3}{0.87\times500}\times\left(0.5 - \frac{1.0\times2500\times10^3}{0.8\times600^2\times35}\right) = 1190 \;\; \text{mm}^2$$

Provide 3 sets of T12 links with T12 cross-ties at 200 mm centres ($A_{sv}$ = 1356 mm$^2$ > $A_{jh}$ = 1190 mm$^2$), thus giving a hoop ratio of joint

$$\frac{A_{sv,j}}{s_{v,j}} = \frac{1356/3}{200} = 2.26$$

From the column section, links in the column non-critical zone are T10@250, thus giving a column hoop ratio

$$\frac{A_{sv,c}}{s_{v,c}} = \frac{314}{250} = 1.26$$

Check the requirement of horizontal transverse joint reinforcement:

$$\frac{A_{sv,j}}{s_{v,j}} = 2.68 > \frac{A_{sv,c}}{s_{v,c}} = 1.26$$

This is satisfactory.

The vertical joint shear force

$$V_{jv} = V_{jh}\frac{h_b}{h_c} = 2055\times\frac{700}{600} = 2397.5 \;\; \text{kN}$$

The area of vertical joint shear reinforcement

$$A_{jv} = \frac{0.4V_{jv} - C_j N^*}{0.87 f_{yv}} = \frac{0.4\times2397.5\times10^3 - 1.0\times2500\times10^3}{0.87\times500} < 0$$

Equation 6.73

Hence, no verical joint shear reinforcement is needed.

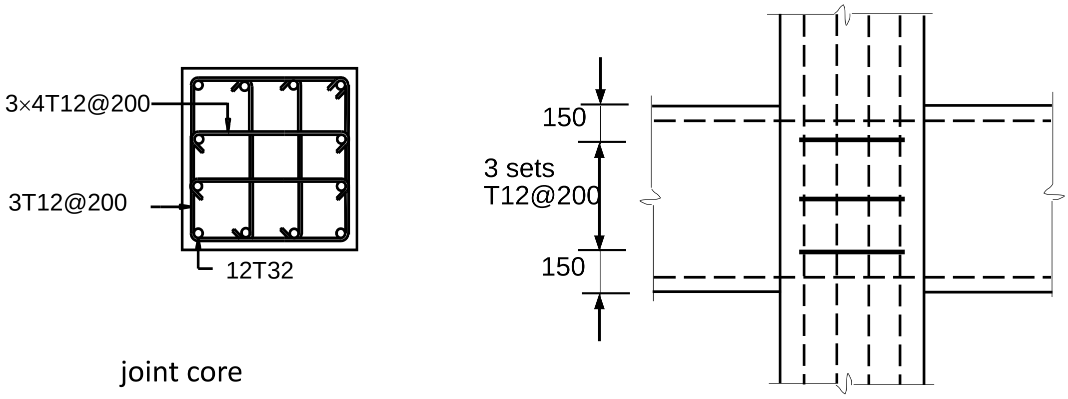



Figure 6.28: Reinforcement details for the interior joint

Worked example 6.10: Design of exterior beam-column joint

Figure 6.29 shows the exterior joint of a reinforced concrete frame designed to resist gravity loads and normal wind loads, with beam and column dimensions and steel reinforcement as indicated. The frame storey height is 3.6 m. The minimum design axial load of the column is 300 kN. Material strengths are $f_{cu}$ = 35 N/mm$^2$ and $f_y$ = 500 N/mm$^2$. Design the joint reinforcement.

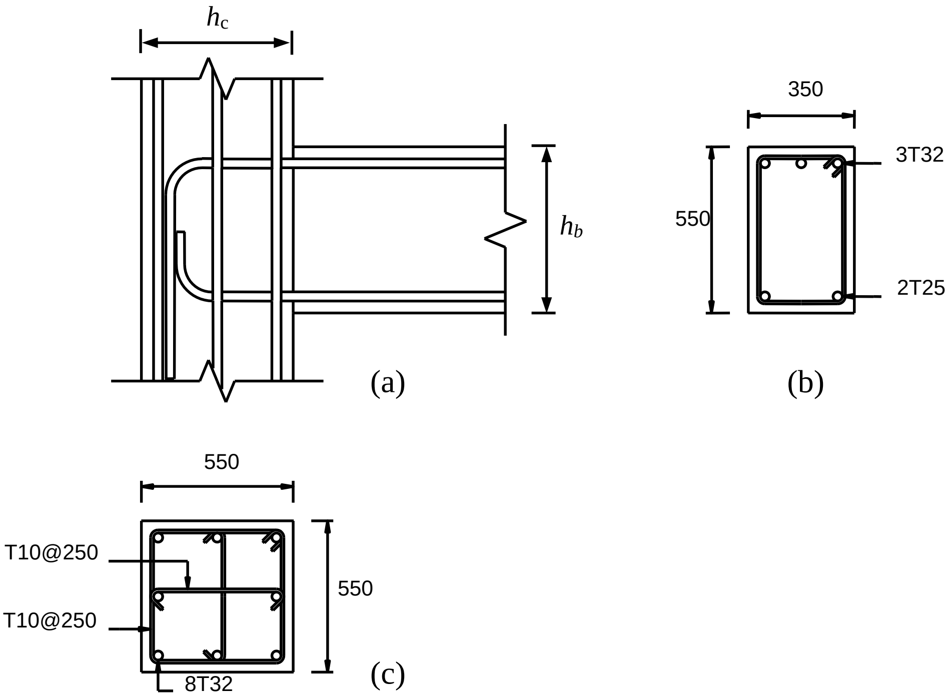



Figure 6.29: Exterior beam-column joint. (a) Beam-column connection; (b) beam cross-section; (c) column cross-section of non-critical zones

The maximum tension force in beam top reinforcing bars is

$$T_{b1} = A_s f_y = 2413 \times 500 \times 10^{-3} = 1206.5 \text{ kN}$$

and

$$T_{b2} = 0 \text{ (an exterior joint)}$$

For equilibrium,

$$A_s f_y = 0.67 f_{cu} b(0.9x)$$

then the neutral axis depth of the beam

$$x = \frac{A_s f_y}{0.9 \times 0.67 f_{cu} b} = \frac{A_s f_y}{0.603 f_{cu} b} = \frac{1206.5 \times 10^3}{0.603 \times 35 \times 350} = 163 \text{ mm}$$

The design moment applied at the join face is determined by

$$M_u = A_s f_y \left( d - \frac{0.9x}{2} \right) = 1206.5 \times 10^3 \times \left( 500 - \frac{0.9 \times 163}{2} \right) = 514.7 \text{ kNm}$$

Column shears corresponding to this joint moment are calculated using the free body of the column between mid-height inflection points (Figure 6.25), given by

$$V_{col} = {}^{M_u}/_{l_c} = {}^{514.7}/_{3.6} = 143.0 \text{ kN}$$

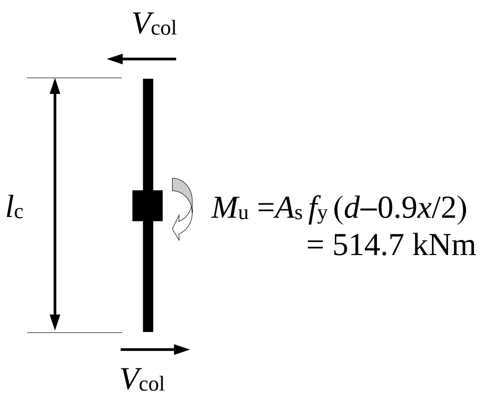



Figure 6.30 Column shear calculated from free body of column between points of contraflexure

The total horizontal joint shear force

$$V_{jh} = T_{b1} - V_{col} = 1206.5 - 143.0 = 1063.5 \text{ kN}$$

Check the horizontal shear stress:

$$\frac{V_{jh}}{b_j h_c} = \frac{1063.5 \times 10^3}{550 \times 550} = 3.52 \text{ N/mm}^2 < 0.2 f_{cu} = 7 \text{ N/mm}^2 \text{, as required.}$$

For a two-dimensional joint, $C_j$ = 1.0. Hence the area of horizontal joint shear reinforcement

$$A_{jh} = \frac{V_{jh}}{0.87 f_{yh}} \left( 0.5 - \frac{C_j N^*}{0.8 A_g f_{cu}} \right) = \frac{1063.5 \times 10^3}{0.87 \times 500} \left( 0.5 - \frac{1.0 \times 300 \times 10^3}{0.8 \times 550^2 \times 35} \right)$$
$$= 1136 \text{ mm}^2$$

Equation 6.72

Provide 3 sets of T16 2-leg link with T10 cross-ties at 150 mm centres ($A_{sv}$ = 1441mm$^2$ > $A_{jh}$ = 1136 mm$^2$), thus giving a hoop ratio of joint

$$\frac{A_{sv,j}}{s_{v,j}} = \frac{1441/3}{200} = 2.40$$

From the column section, hoops in the column non-critical zone are T10@250, thus giving a column hoop ratio

$$\frac{A_{sv,c}}{s_{v,c}} = \frac{235}{250} = 0.94$$

Check the requirement of horizontal transverse joint reinforcement:

$$\frac{A_{sv,j}}{s_{v,j}} = 2.4 > \frac{A_{sv,c}}{s_{v,c}} = 0.94$$

This is satisfactory.
The vertical joint shear force

$$V_{jv} = V_{jh} \frac{h_b}{h_c} = 1063.5 \times \frac{550}{550} = 1063.5 \text{ kN}$$

The area of vertical joint shear reinforcement

$$A_{jv} = \frac{0.4V_{jv} - C_j N^*}{0.87 f_{yv}} = \frac{0.4 \times 1063.5 \times 10^3 - 1.0 \times 300 \times 10^3}{0.87 \times 500} = 288.3 \text{ mm}^2$$

Equation 6.73

Provide two intermediate bars (2T12) at each vertical face of the joint. The total area provided is 452 mm$^2$. The arrangement of joint reinforcement is shown in Figure 6.31.

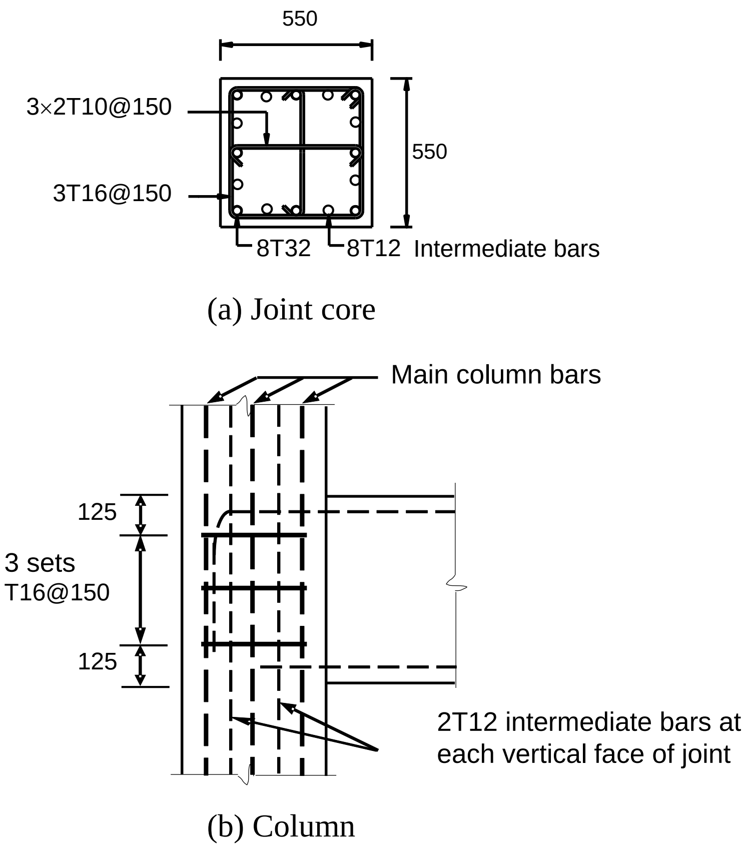



Figure 6.31 Reinforcement details of the exterior joint

# 7 SERVICEABILITY LIMIT STATES

## 7.1 General

The design requirements of serviceability limit state may be satisfied by:

- following the deemed-to-satisfy rules in dimensioning and detailing of the structural elements; or
- analysing the loading effects and verifying that the deformations and crack widths etc are within acceptable limits (this is often referred to as direct calculation in the code).

When analysing the structure for satisfying the serviceability limit state requirements, the most likely behaviour of the structure should be modelled. During analysis of structure in ultimate limit state design, simplifying assumptions that are on the safe side but not necessarily realistic are often made. For instance, in ultimate limit state design, it is often assumed that the torsional stiffness of frame members, the out-of-plane stiffness of shear walls and the stiffness of non-structures are negligible. Such assumptions are conservative for ultimate limit state design but are not necessarily so when applied to serviceability limit state design. The existence in reality of the stiffness that has been neglected could significantly affect the internal load distribution within the structure thereby causing unexpected cracking of certain structural components whose loading effects have been underestimated. Hence, during analysis of structure in serviceability limit state design, the structural model should be as close to the reality as possible; no stiffness of any structural or non-structural component should be neglected if its presence could affect the internal load distribution. If the structural parameters of certain components, especially those of non-structural components, are difficult to be determined accurately, then at least the possible ranges should be estimated and the analysis of structure carried out repeatedly using different combinations of possible values of the structural parameters so as to allow for the worst possible scenario in the design.

One may argue why we have to take the above trouble just for serviceability limit state design. Many structural engineers regard ultimate limit state design as much more important than serviceability limit state design and tend to pay less attention to cracking, deflection and vibration of the structure. In actual fact, after decades of massive building development in many cities over the world, the design and construction of a concrete building satisfying the ultimate limit state requirements no longer present any major difficulties. However, we are still having lots of serviceability problems, which arise mostly from cracking of concrete. The author has been serving as an independent consultant on concrete materials and structures for many years. In all the problematic cases that the author has investigated, although the developers and the owners/tenants were complaining about the serious cracking of the buildings within several years after occupation and were deeply concerned with the safety of the buildings, there was in every case no structural safety problem at all. All the problems were due to inadequate serviceability limit state design. The argument of some structural engineers that concrete structures are expected to crack and therefore cracking is normal and should be acceptable is not entirely defensible. In many cases, the extent of cracking and the crack widths could have been controlled to be much smaller if proper considerations had been given during the serviceability limit state design.

The author has been advocating the idea of designing "high-performance concrete buildings", which we all should strive for. A high-performance concrete building is one that has, not just a high safety standard, but also all round high performance in terms of serviceability, functionality, durability, structural assurance measures, reparability, environmental friendliness and sustainability. All in all, the author holds the strong view that apart from safety, the other performance attributes such as those mentioned above, especially serviceability, should also be carefully attended to in the structural design, as promulgated in the following paper.
Kwan A.K.H., Au F.T.K. and Lee P.K.K., "High-performance concrete buildings for the new millennium", Progress in Structural Engineering and Materials, Vol.5, No.4, October-December, 2003, pp263-273.

When assessing the loads in the analysis of structure for serviceability limit state design, a distinction should be made between *characteristic loads* and *expected loads*. Characteristic loads are conservative estimates of the loads that could occur with a relatively long return period while expected loads are the likely loads that could occur every now and then with a relatively high frequency of occurrence. Generally, for estimating the maximum response of the structure under normal conditions (the code calls this the limit state calculation or calculation to satisfy a particular limit state), the characteristic loads should be used, but for estimating the likely or average response of the structure under normal conditions (the code calls this the best estimate calculation), the expected loads should be used instead. To clarify why "under normal conditions" is emphasized above, the author would like to stress that the estimation of maximum response under abnormal or extreme conditions is part of ultimate limit state design, not serviceability limit state design. For dead loads, the characteristic and expected values are the same and thus it will be sufficient to just use the characteristic values in the analysis. For imposed loads, however, both the characteristic and expected values may have to be considered separately.

When calculating deflection, it is necessary to assess how much of the applied load is permanent and how much is transitory. Dead load is 100% permanent. But the proportion of the characteristic imposed load that should be considered as permanent is dependent on the usage of the building. The code suggests that for normal domestic or office occupancy, 25% of the characteristic imposed load should be considered as permanent and for structures used for storage, at least 75% should be considered as permanent when the upper limit to the deflection is being assessed. This suggestion is based on BS8110: Part 2: 1985.

When analysing the structure for serviceability limit state design, the elastic modulus of concrete should be taken as the value given in Table 3.2 or evaluated using Equation 3.1. The footnote of Table 3.2 says that where the mean or characteristic value of elastic modulus is required, the appropriate mean or characteristic strength of the concrete should be used for the evaluation. For estimating the maximum response of the structure under normal conditions, the elastic modulus corresponding to the *characteristic concrete strength* should be used, but for estimating the average response of the structure under normal conditions, the elastic modulus corresponding to the *expected concrete strength* (i.e. the mean strength) should be used instead. If there is any uncertainty in the elastic modulus, the possible range of values should be estimated and different values within the range tried in the analysis to obtain an idea of the reliability of the calculation.

Regarding the method of analysis for serviceability limit state design, a linear elastic analysis method may be used. Three methods of evaluating the section stiffness of the members have been given in Clause 5.1.2 of the code. They are all regarded as acceptable but a consistent method should be applied to all members of the structure. Where a single value of section stiffness is used to characterise a member, it should be sufficiently accurate to evaluate the section stiffness based on the uncracked concrete section (the entire concrete section, ignoring the reinforcement) even though the calculation shows the members to be cracked. Where more sophisticated methods of analysis are used in which variations in properties over the length of members can be taken into account, it might be more appropriate to calculate the stiffness of the highly stressed parts of the members based on the cracked transformed section (the compression area of the concrete section combined with the reinforcement on the basis of modular ratio).

## 7.2 Cracking

### 7.2.1 General

There are three main reasons for controlling cracking of a reinforced concrete structure:

- ensure proper functioning;
- avoid impairing durability;
- avoid causing unacceptable appearance.

Depending on the location and the lighting condition, a crack of width greater than 0.3 mm can be quite conspicuous even to someone standing at a distance of several metres away. Hence, for aesthetic reason, there is a need to control the crack width at not greater than 0.3 mm. If the cracking is extensive and at the same time the crack widths are relatively large (> 0.3 mm), the tenants or owners may feel uneasy about the safety of the building and start complaining (even though from the structural engineer's point of view, the building is still very safe). Hence, in order to safeguard ourselves from future complaints and to avoid imparting the bad impression to the public that the building has not been properly designed, it should be prudent to pay particular attention to crack control. Failure to control cracking would not in general cause any immediate safety problem but would in the longer term create mistrust between the general public and the structural engineering profession.

The code states, in Clause 7.2.1, that “cracking is normal in reinforced concrete structures”. This statement is actually extracted from Eurocode 2 (there is no similar statement in BS8110). However, our clients, the future tenants and the general public are not necessarily in agreement with this statement. Young engineers may also misinterpret this statement as suggesting that crack control is unimportant. Therefore, the author would like to elaborate and qualify this statement as “minor cracking in reinforced concrete structures is normal but extensive cracking should be avoided”.

The crack width limits are given in Table 7.1 of the code. Basically, for prestressed members with bonded tendons, the *frequent load combination* should be considered and the crack width should be limited to 0.2 mm, and for reinforced members and prestressed members with unbonded tendons, the *quasi-permanent load combination* should be considered and the crack width should be limited to 0.3 mm (except water retaining structures for which a lower limit of 0.2 mm needs to be applied).

The load combinations to be considered, namely, frequent load combination and quasi-permanent load combination, are adopted from Eurocode 2. According to Eurocode 0: Basis of Structural Design (also called EN1990), a frequent load combination comprises of the permanent actions and the frequent values of the variable actions, whereas a quasi-permanent load combination comprises of the permanent actions and the quasi-permanent values of the variable actions. The frequent value of a variable action is the value that the total period of time for which it will be exceeded is a small fraction of the reference period, while the quasi-permanent value of a variable action is the value that the total period of time for which it will be exceeded is a large fraction of the reference period (see Clause 1.5.3 of Eurocode 0). They are both given as reduced values of the respective characteristic values; the frequent value is given as $\Psi_1$ times the characteristic value ($\Psi_1 < 1$) while the quasi-permanent value is given as $\Psi_2$ times the characteristic value ($\Psi_2 < 1$). These terms are not entirely compatible with the terminology used in the British Standards. The author would like to suggest that the frequent value of a variable load may be interpreted as the expected high load under normal condition (that could occur frequently and might be exceeded for a small fraction of time), whereas the quasi-permanent value of a variable load may be interpreted as the expected average load under normal condition (that would be in place most of the time and could be exceeded for a substantial fraction of time). In other words, they are in reality expected loads under normal condition with different frequencies of occurrence.

Annex A1 of Eurocode 0 has provided the values of $\Psi_1$ and $\Psi_2$ for imposed loads in buildings as in Table 7.1. These values are not in line with what have been suggested in Clause 7.1.3.3 of the code (this clause is actually the same as the corresponding clause in BS8110: Part 2: 1985). Basically, the values given in Eurocode 0 are marginally higher than those suggested in BS8110: Part 2. As the design and calculation methods in the new code follow those in BS8110, it is considered more appropriate to adopt the values given in BS8110 because the design/calculation methods and the design values together form an integrated package and should be compatible with each other. Adjusting the values of $\Psi_1$

and $\Psi_2$ provided by Eurocode 0 slightly so that they become compatible with BS8110, the values of $\Psi_1$ and $\Psi_2$ to be used in the new code may be obtained as in Table 7.2.

Table 7.1 Values of $\Psi_1$ and $\Psi_2$ for imposed loads given in Eurocode 0

| Building area over which imposed load is applied | $\Psi_1$ | $\Psi_2$ |
|---|---|---|
| Domestic or office areas | 0.50 | 0.30 |
| Congregation and shopping areas | 0.70 | 0.60 |
| Storage areas | 0.90 | 0.80 |
| Traffic areas, vehicle weight ≤ 30 kN | 0.70 | 0.60 |
| Traffic areas, 30 kN ≤ vehicle weight ≤ 160 kN | 0.50 | 0.30 |
| Roofs | 0.00 | 0.00 |

Table 7.2 Values of $\Psi_1$ and $\Psi_2$ for imposed loads to be used in the new code

| Building area over which imposed load is applied | $\Psi_1$ | $\Psi_2$ |
|---|---|---|
| Domestic or office areas | 0.50 | 0.25 |
| Congregation and shopping areas | 0.70 | 0.60 |
| Storage areas | 0.90 | 0.75 |
| Traffic areas, vehicle weight ≤ 30 kN | 0.70 | 0.60 |
| Traffic areas, 30 kN ≤ vehicle weight ≤ 160 kN | 0.50 | 0.30 |
| Roofs | 0.00 | 0.00 |

### 7.2.2 Control of cracking without direct calculation (deemed-to-satisfy)

Clause 7.2.2 of the code suggests that if the deemed-to-satisfy rules in the code with respect to minimum reinforcement areas and bar spacings are followed, then *usually* no direct calculation needs to be carried out to check compliance with the crack width limits. As an alternative to following the deemed-to-satisfy rules, direct calculation may be carried out to check compliance with the crack width limits. According to Clause 7.2.1 of the code, in most cases, direct calculation to check compliance with the crack width limits would allow wider bar spacings to be used, especially for shallow members.

### 7.2.3 Assessment of crack widths

Only the crack width under long-term condition needs to be assessed. The loads to be applied should be those of the frequent load combination (for prestressed members with bonded tendons) or the quasi-permanent load combination (for reinforced members and prestressed members with unbonded tendons), while the elastic modulus of the concrete should be the long-term modulus taken as one-half of the short-term modulus.

The formula for evaluating the surface crack width has been given as follows:

$$\text{surface crack width } = \frac{3a_{\text{cr}}\varepsilon_m}{1+2\left(\frac{a_{\text{cr}}-c_{\min}}{h-x}\right)} \qquad \text{Equation 7.1}$$

in which $a_{\text{cr}}$ is the distance from the point being considered to the surface of the nearest longitudinal bar, $\varepsilon_m$ is the average strain at the level of the point being considered, $c_{\min}$ is the minimum distance from the surface of concrete to the surface of the tension steel (the cover to the tension steel), *h* is the overall depth of the section and *x* is the depth to neutral axis.

This formula is applicable only when the strain in the tension reinforcement is limited to ${0.8f_y}/{E_s}$, which is generally satisfied at serviceability limit state.

The average strain $\varepsilon_m$ may be evaluated from the curvature calculated as per Clause 7.3.6 of the code based on the following assumptions:

- plane sections remain plane;
- the reinforcement is elastic with an elastic modulus of 200 kN/mm$^2$;
- the concrete in compression is elastic;
- the tensile strength of concrete is not negligible and there is a long-term tensile stress varying linearly from 0 N/mm$^2$ at the neutral axis to 0.55 N/mm$^2$ at the level of the tension reinforcement within the tension zone.

Alternatively, as an approximation, the average strain may be evaluated by first calculating the average strain of a cracked section with the tensile strength of concrete neglected and then reducing this average strain value to take into account the stiffening effect of the concrete in the tension zone.

Worked Example 7.1: Estimation of maximum crack width

Estimate the maximum flexural crack widths for the beam section in a domestic building shown in Figure 7.1. The simply supported beams are spaced at 3 m and the span length is 6 m. The finishes is 1 kPa and imposed load is 2 kPa. Material properties: $f_{cu}$ = 30 N/mm$^2$; $E_c$ = 22.2 kN/mm$^2$, $f_y$ = 500 N/mm$^2$; $E_s$ = 200 kN/mm$^2$

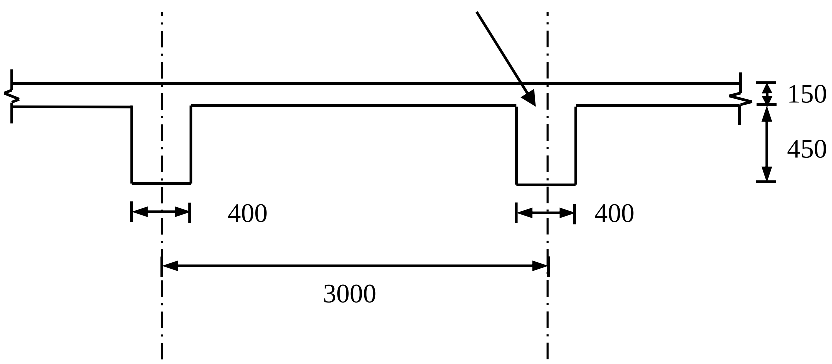



Figure 7.1 Worked example – maximum crack width estimation

| | | | | | |
|---|---|---|---|---|---|
| Loading | D.L. | S.W. (slab) | $0.15 \times 24.5 \times 3$ | = | 11.03 kN/m |
| | | S.W. (beam) | $0.4 \times 0.45 \times 24.5$ | = | 4.41 kN/m |
| | | Fin | $1 \times 3$ | = | 3.0 kN/m |
| | | | | = | 18.44 kN/m |
| | L.L. | | $2 \times 3$ | = | 6.0 kN/m |

Ultimate Moment = $(1.4\times18.44+1.6\times6)6^2/8 = 159.4$ kNm

Design for ultimate state,

d = 600 – 40 – 10 = 550 mm (Assume cover to be 40 mm and T20 is used)

$\frac{M}{f_{cu}bd^2} = \frac{159.4 \times10^6}{30\times400\times550^2} = 0.044$, z = 0.948d = 521.4 mm

$A_s = 703$ mm$^2$

Provide 3T20 (Area provided is 942 mm$^2$, 0.39% > 0.13%)

Crack width is checked by (7.1) and (7.2)
To calculate crack width, it is first necessary to assess the neutral axis depth x by the elastic theory

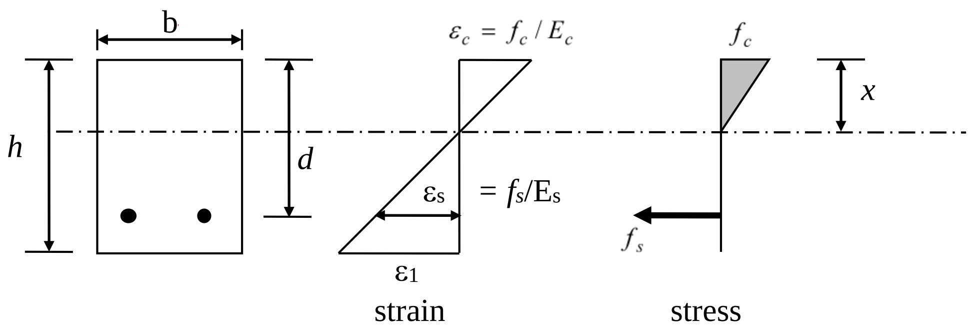



Figure 7.2 Stress/strain relation of a cracked R.C. section

According to Clause 7.2.3, the modulus of elasticity of the concrete should be taken as half the instantaneous values: $E_c = 22.2/2 = 11.1$ kN/mm$^2$

Consider equilibrium of the section

$$\frac{1}{2} f_c bx = f_s A_{st} \Rightarrow \frac{1}{2} E_c \varepsilon_c bx = E_s \frac{\varepsilon_c (d-x)}{x} A_{st}$$

$$\frac{1}{2} bx^2 + \alpha_e A_s x - \alpha_e A_s d = 0 \quad \text{where} \quad \alpha_e = \frac{E_s}{E_c} = 18$$

$$x = \frac{-\alpha_e A_s \pm \sqrt{(\alpha_e A_s)^2 + 2b\alpha_e A_s d}}{b} = \frac{-(18)(942) \pm \sqrt{(18\times942)^2 + 2(400)(18)(942)(550)}}{400}$$

Solving $x = 177.67$ mm

For checking crack width, quasi-permanent moment is considered.

M = (18.44 + 0.25×6)$6^2$ /8 = 89.73 kNm  ($\psi_2$ = 0.25 for domestic area)

Taking moment about the centroid of the triangular concrete stress block, the steel tensile stress can be worked out as

$M = f_s A_s \left(d - \frac{x}{3}\right)$, by arranging the terms,

$$f_s = \frac{M}{A_s\left(d - \frac{x}{3}\right)} = \frac{89.73 \times 10^6}{942\left(550 - \frac{177.67}{3}\right)} = 194.1 \text{ N/mm}^2$$

So the strain of the steel is $\varepsilon_s$ = 194.1/(200×$10^3$) = 0.00097< 0.8$f_y$/$E_s$ = 0.002

At the level of the concrete tension side, the strain is

$$\epsilon_1 = \epsilon_s \frac{(h - x)}{(d - x)} = 0.00097 \frac{600 - 177.67}{550 - 177.67} = 0.0011$$

By (Eqn 7.2),

$$\epsilon_m = \epsilon_1 - \frac{b_t(h - x)(a' - x)}{3E_s A_s(d - x)}$$

$$= (0.0011) - \frac{(400)(600 - 177.67)(600 - 177.67)}{3 \times 200 \times 10^3 \times 942 \times (550 - 177.67)} = 0.00076$$

The maximum crack width may occur at several locations (1, 2 and 3). In this example, it is assumed that the maximum crack width occurs at the bottom of the beam. In practice, location (3) with equidistance between the neutral axis and the bar surface should also be checked.

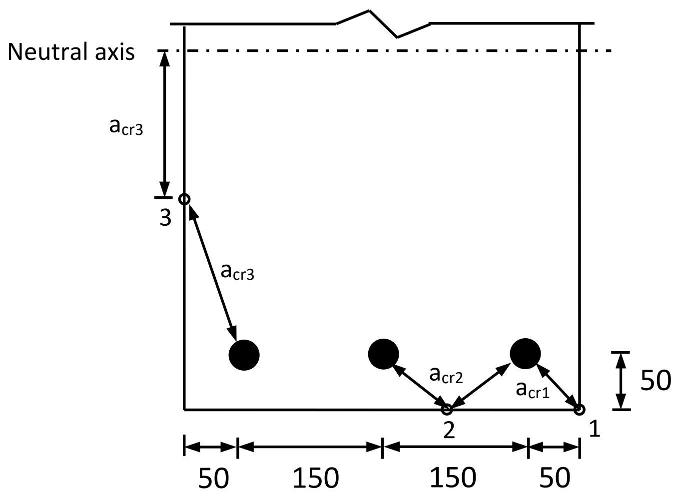


Figure 7.3 Locations for maximum crack width

In this example, $a_{cr1} = \sqrt{50^2 + 50^2} - 10 = 60.7$ mm and

$$a_{cr2} = \sqrt{50^2 + 75^2} - 10 = 86 \text{ mm}$$

By (Eqn 7.1) the maximum cracked width ($w_{max}$) is

$$w_{max} = \frac{3a_{cr}\epsilon_m}{1 + 2\left(\frac{a_{cr} - c_{min}}{h - x}\right)} = \frac{3(86)(0.00076)}{1 + 2\left(\frac{86 - 40}{600 - 177.67}\right)} = 0.16 \ mm$$

The estimated size of maximum crack width is smaller than 0.3 mm.

Where it is expected that the concrete may be subjected to abnormally high shrinkage ($> 0.0006$), the value of $\varepsilon_m$ should be increased by adding 50% of the expected shrinkage strain (this requirement is based on BS8110: Part 2: 1985). From Clause 3.1.3 of the code, it can be seen that the shrinkage of concrete in Hong Kong is in general much higher than that of concrete in UK. The author is at the moment conducting a fairly large number of shrinkage tests on typical normal-strength concrete, high-strength concrete and high-performance concrete in Hong Kong. The tests are not completed yet but judging from the test results obtained so far, it may be roughly estimated that at a relative humidity of 50%, the shrinkage strain ranges from 0.0007 to 0.0009, while at a relative humidity of 75%, the shrinkage strain ranges from 0.0006 to 0.0008. It is now almost certain that the shrinkage of concrete in Hong Kong is generally higher than 0.0006. Hence, the value of $\varepsilon_m$ will have to be increased by 50% of the expected shrinkage strain. Assuming the expected shrinkage strain to be 0.0006, the value of $\varepsilon_m$ will be increased by 0.0003 (0.0006 × 50%). This increase may amount to more than 30% of the original value of $\varepsilon_m$ and is definitely not negligible. The omission of the effect of shrinkage in serviceability limit state design in the past is one of the reasons for having, from time to time, concrete cracking problems in Hong Kong. Inclusion of this requirement in the new code would significantly improve the crack control of concrete structures.

It should be noted that with the effect of shrinkage taken into account, the statement in Clause 7.2.1 of the code, which says that in most cases, direct calculation to check compliance with the crack width limits would allow wider bar spacings to be used, especially for shallow members, is no longer valid. In fact, with the effect of shrinkage taken into account, the crack control requirements set for direct calculation of crack width would become more stringent than those imposed through the deemed-to-satisfy rules. Because of this, Clause 7.2.2 needs more careful interpretation. Where the shrinkage of concrete is higher than 0.0006, it may not be good enough to just follow the deemed-to-satisfy rules; in such case, it is better to always carry out direct calculation to check compliance with the crack width limits as per Clause 7.2.3.

During the course of development of the new code, a comparison between the requirements of BS8110 and Structures Design Manual for Highways and Railways has been made. The Structures Design Manual has more onerous crack width limits, which are dependent on the exposure conditions. Generally, the maximum allowable crack width varies from 0.10mm for extreme exposure conditions to 0.25 mm for moderate exposure conditions. In this regard, it should be noted that the design life of highway/railway structures is 120 years whereas the design life of normal buildings is only 50 years. A comparison of the various crack width formulas given in BS8110, Eurocode 2 and GB50010 by calculating the crack widths of a typical beam section has also been made. The results are quite varying and therefore cannot be compared directly. It should, however, be noted that the treatment of crack width must be carried out in conjunction with the complete code of practice philosophy, including the loading intensities to be adopted, the material stresses to be used, the concrete cover to be specified and the tolerable crack widths etc. It will be dangerous to single out the crack width formula out of context. Crack control is not an exact science because cracking is a semi-random phenomenon and many aspects of concrete cracking are still not fully understood. Rigorous theoretical treatment is not possible at this stage. The effectiveness of the proposed crack control measures will have to be evaluated through regular review of the experience gained from field applications.

### 7.2.4 Early thermal cracking

Early thermal cracks are caused by internal or external restraints against the thermal movement of the concrete as the temperature of the concrete changes due to the heat generated from the chemical reactions of the cementitious materials. Internal restraint is the major cause of cracking in massive concrete structures, while external restraint is the major cause of cracking in concrete structures cast against movement restraints.

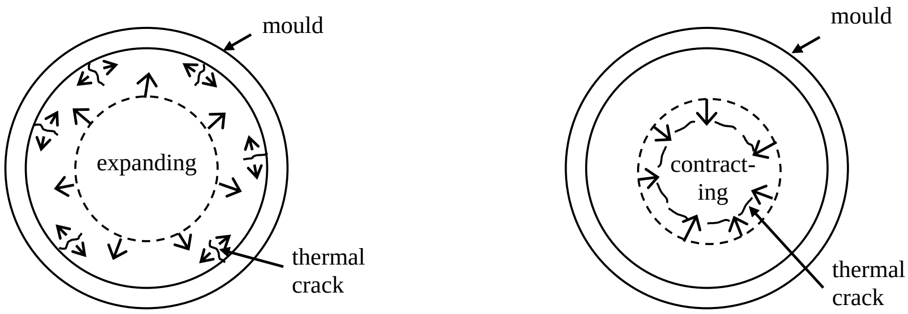
To assess the risk of early thermal cracking, three issues need to be considered:

- measuring or estimating the adiabatic temperature rise (the temperature rise with no heat loss) of the concrete mix during curing;
- analysing the temperature distribution within the concrete mass taking into account heat generation by the concrete and heat loss to the environment;
- analysing the thermal strain induced by internal and external restraints.

Under adiabatic condition, the hydration of cement would produce a temperature rise of about 12°C per 100 kg/m$^3$ of cement regardless of the type of cement used. Partial replacement of cement by pulverized fuel ash (PFA) could reduce the amount of heat generated. The temperature rise due to the pozzolanic reaction of PFA is dependent on the temperature, but is generally lower than that due to the hydration of cement. For instance, at a temperature of 60°C, the pozzolanic reaction of PFA would produce a temperature rise of only 5.5°C per 100 kg/m$^3$ of PFA. More information on the estimation of temperature rise can be found in the following publications:
FitzGibbon M.E., "Large pours – 2, heat generation and control", Concrete, 10(12), 1976, pp33-35.
Building Research Establishment, Technical Guide on Control of Early Thermal Cracking in Concrete, PSA Specialist Services, UK, 1992, 14pp.

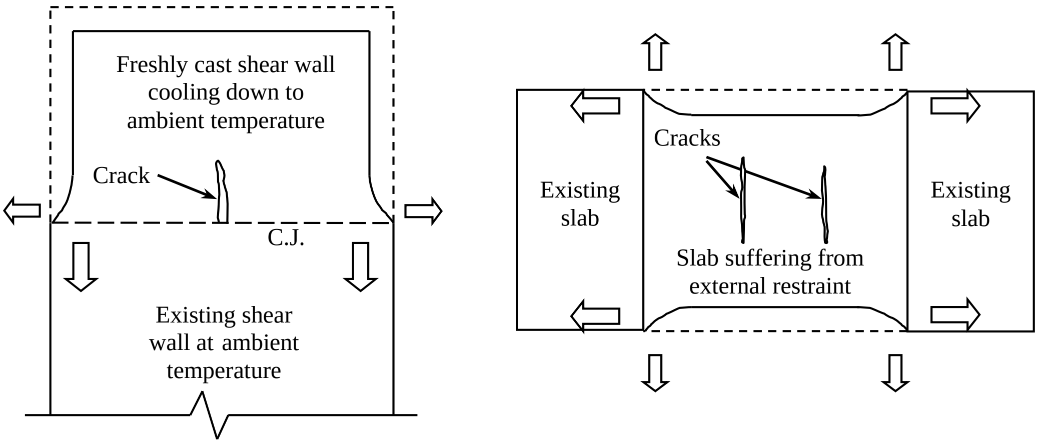
The temperature distribution within the concrete mass is dependent on both the heat generation from the concrete causing temperature rise and the heat dissipation through the boundary surfaces causing temperature drop and temperature gradient within the concrete. Taking into account the various factors determining the heat generation and dissipation, the temperature distribution and variation with time may be evaluated for members with simple geometry using an analytical method and for members with complex geometry by the finite element method. However, even with the temperature distribution known, the evaluation of the tensile stresses developed within the concrete is not an easy task. This is partly because apart from the external restraint against thermal movement, the concrete itself can also act as an internal restraint against differential thermal movement arising from uneven temperature distribution and partly because the restraint-induced stresses are dependent also on the changing elastic modulus and creep characteristics of the concrete at early age. In fact, the mechanisms of how the internal and external restraints cause tensile stresses and thermal cracking are often misunderstood, leading to inappropriate practices during curing that often further aggravate the thermal cracking problem, as explained in the following.

*Internal restraint:*
Due to heat dissipation, the temperature of the concrete near the surfaces is lower than that of the concrete at the centre. The resulting temperature difference may induce tensile strains large enough to cause thermal cracking within the surface zones when the temperature is rising and in the interior when the temperature is dropping, as depicted in Figure 7.4. When the temperature is rising, the concrete at the centre has a higher temperature and its thermal expansion will induce bursting pressure onto the outside causing the concrete within the surface zones to be subjected to tension and possibly cracking. The thermal cracks formed within the surface zones can be observed after removal of formwork and are thus well known. When the temperature is dropping, the concrete near the surfaces will cool down to ambient temperature first while the concrete at the centre will continue to cool down. Cooling of the concrete in the interior within a bigger mass of concrete will induce tensile strains that may cause cracking. Since the thermal cracks formed in the interior cannot be seen unless coring is carried out, many engineers are not aware of this problem. As the movement restraint that induces these cracks is from the concrete mass itself, it is called internal restraint.




(a) when temperature is rising (b) when temperature is dropping

Figure 7.4: Thermal cracks induced by internal restraint




(a) edge restraint (b) end restraint

Figure 7.5: Thermal cracks induced by external restraint

*External restraint:*
Subsequent temperature drop after the concrete has reached the peak temperature causes thermal contraction. If the thermal contraction is restrained by existing structures bonded to the freshly cast concrete, as shown in Figure 7.5, tensile stresses will be induced and thermal cracks, which are generally through cracks, may be formed. As the movement restraint is from outside, it is regarded as an external restraint. There are two types of external restraint: edge restraint and end restraint. Edge restraint is quite common when a shear wall is constructed storey-by-storey in which case the shear wall at the lower storey restrains the thermal movement of the freshly cast concrete at one edge, while end restraint usually occurs when a wall or slab is constructed bay-by-bay in which case the existing walls or slabs restrain the thermal movement of the freshly cast concrete at two opposite ends. The author had once encountered a thermal cracking problem, which occurred when a shear wall was cast using high-strength concrete. During curing, the temperature of the concrete reached a peak of around 80°C. After the mould was removed, fairly wide thermal cracks were observed. The contractor

had tried to resolve the problem by providing better insulation but the problem persisted and in fact was worsened due to reasons explained below.

*Provision of insulation:*
There is a common misunderstanding that insulation is a proper way to deal with thermal cracking, regardless of the site conditions. In some contracts, it is even stipulated in the specification that double-layer insulation is to be provided in all cases when curing freshly cast concrete. Actually, it may help to avoid thermal cracking in some cases but may also aggravate the problem in other cases, depending on the type of restraint. If there is no external restraint, then the most likely cause of thermal cracking is internal restraint. In this case, insulation should be applied. It could reduce the temperature difference within the concrete and thereby alleviate the thermal cracking problem. However, it should be noted that the insulation and also the formwork should not be removed until the temperature of the concrete has dropped to near ambient temperature. Moreover, after removal of formwork, no water spraying onto the concrete surfaces should be applied. It is not uncommon in Hong Kong that the formwork is removed while the concrete is still hot and cold water is then sprayed onto the concrete for curing, leading to temperature shock and immediate cracking. If there is external restraint, then it is the thermal contraction of the concrete during temperature drop that will be the major cause of thermal cracking. In such case, insulation would increase the temperature rise of the concrete and of course also the subsequent temperature drop, thereby aggravating the thermal cracking problem. Therefore, insulation should not be applied when there is any external restraint. Reducing the heat generation by changing the mix design of the concrete would help but if the situation remains critical passive or active cooling might have to be applied. If there are both internal and external restraints, as in the case of a thick concrete member to be cast against existing structures that will restrain its thermal movement, then it is not a question of whether to insulate or not to insulate. The only acceptable method is to reduce both the peak temperature of the concrete and the temperature differential within the concrete. One feasible solution is to provide active internal cooling so as to draw out heat directly from the interior of the concrete. Lowering the placing temperature of the concrete and reducing the heat generation of the concrete should also be considered.

*Mix design:*
It is advantageous for a concrete structure to have a small temperature rise during curing. A small temperature rise ensures minimal thermal movement and reduces the risk of early thermal cracking. In many cases, it is possible to alter the mix proportion of the concrete mix without affecting its rheological properties and desired compressive strength to reduce the heat generation of the concrete so as to keep the temperature rise as small as possible. This is especially so for today's concrete, which usually contains a fairly high cementitious materials content. The heat generation from a concrete mix is affected mainly by the quality and quantity of cement, mineral admixtures and chemical admixtures. If cement is the only cementitious material in the concrete mix, its hydration contributes to all the heat generated and the temperature rise would be proportional to the cement content. The fineness of the cement may also affect the temperature rise; a finer cement hydrates and generates heat at faster rates and, with relatively little time to dissipate the heat, the resulting temperature rise would be higher. Replacing part

of the cement by PFA can significantly reduce the amount of heat generated. This is because the pozzolanic reaction of PFA does not start until the alkalinity of the pore water is high enough to start the reaction, which requires part of the cement to hydrate for releasing calcium hydroxide into the pore water. On the other hand, the effect of replacing part of the cement by condensed silica fume (CSF) on heat generation is more complex. CSF provides nucleation sites for hydration and would thus speed up the heat generation due to hydration of cement. Moreover, since CSF is much finer than PFA, the pozzolanic reaction of CSF is faster than that of PFA. As the increase in heat generation due to addition of CSF could outweigh the decrease in heat generation resulting from reduction in cement content, the replacement of part of the cement by an equal weight of CSF might not help to reduce the temperature rise. Lastly, the addition of a superplasticizer can improve the rheology of the paste and reduce the water demand of the concrete mix, but generally has little effect on the heat generation of the cementitious materials. Nevertheless, the addition of a superplasticizer does have an indirect effect on heat generation. Whilst the superplasticizing effect does not seem to affect heat generation much, it reduces the water demand of the concrete mix and thus with a superplasticizer added the paste volume of the concrete mix could be reduced while maintaining the same workability. After reduction of the paste volume, there should be pro-rata reductions in the cementitious materials content and the amount of heat that would be generated.

*Adiabatic curing test:*

Although simple rules for predicting temperature rise are available (such as BRE's technical guide), they are only rules of thumb and are not expected to be accurate. In fact, the temperature rise is dependent on the material characteristics and the local conditions, and thus the only reliable way of estimating the temperature rise is to carry out adiabatic curing test of the concrete mix using exactly the same constituent materials. The purpose of the adiabatic curing test is to simulate the adiabatic condition (i.e. no heat gain or loss condition) inside a large volume of concrete and measure the temperature rise of the concrete under such condition. From the test, the variation of the concrete temperature with time can be obtained, based on which the rate of temperature rise at different times after casting and the total temperature rise may be evaluated. These data are needed for finite element analysis of the temperature and thermal stress distributions within the concrete, the outcome of which would help to design a proper thermal control scheme for mitigating early thermal cracking. Although the principles of adiabatic curing test are well founded, there is still no generally accepted or standardized test method. As a result, the details of the test vary from one laboratory to another. Some researchers use a computer controlled environmental chamber to simulate the adiabatic condition by automatically adjusting the temperature of the chamber to be constantly equal to the temperature at the centre of the concrete mass as measured by thermocouples buried into the concrete, while others just cure the concrete specimen and measure the temperature at the centre of the specimen using thermocouples without providing any kind of insulation or temperature control. Frankly speaking, if the adiabatic curing conditions are not exactly simulated, the curing test is not a true adiabatic curing test and should be called a temperature rise evaluation test (TRET). In Hong Kong, adiabatic curing tests are often carried out without exercising any temperature control; these should have been more appropriately referred to as TRET. However, the adiabatic curing test is more suitable for use in a well-equipped laboratory and for field tests the less sophisticated TRET methods are preferred.

*Thermal analysis and control:*
In the past few years, the author has investigated a number of thermal cracking problems in Hong Kong and found that in virtually all cases, the thermal cracking was associated with high cementitious materials content of the concrete mix and high temperature rise of the concrete during curing. It is therefore suggested that if the cement content exceeds 450 kg/m$^3$ or the total cementitious materials content exceeds 550 kg/m$^3$, the contractor should be required to carry out an adiabatic curing test or a TRET. Then, if the temperature rise of the concrete as measured by a true adiabatic curing test is higher than 45$^o$C or as measured by an acceptable TRET is higher than 40$^o$C (TRET tends to yield a lower temperature rise because of heat loss), the contractor should be required to carry out thermal analysis and submit a proposal for thermal control of the curing concrete.

## 7.3 Deformations

### 7.3.1 General considerations

There are several reasons for controlling deformation of a structure:

- avoid causing unacceptable appearance;
- ensure proper functioning;
- avoid causing drainage problems;
- avoid causing damage to adjacent structures;
- avoid causing damage to non-structures such as finishes/partitions/fixings.

The deformation control requirements may be satisfied by either following the deemed-to-satisfy rules with respect to maximum span/effective depth ratio given in Clause 7.3.4 without carrying out any direct calculation or by direct calculation of the deflection of the structure according to Clause 7.3.5 and checking of the deflection results against the permitted values.

To avoid impairing the appearance and general functioning of the structure, the *permanent sag* (the total long-term vertical deflection under quasi-permanent loads) should not exceed span/250. Pre-cambering may be used to compensate for some or all of the permanent sag, but any pre-cambering should not be larger than span/250.

To avoid causing damage to adjacent structural or non-structural parts of the structure, the long-term vertical *deflection after construction* under quasi-permanent loads should not exceed span/500. It should be noted that it is the deflection of the structure after the adjacent parts are connected to the structure (i.e. after construction) that causes damage to the adjacent parts; the deflection of the structure before the adjacent parts are connected to the structure has no effect. Hence, pre-cambering would not help to avoid causing damage to the adjacent parts. It should also be noted that although the dead loads are usually applied before the adjacent parts are connected to the structure, the creep of the structure under dead loads could contribute to deflection of the structure after the adjacent parts are connected to the structure. In reality, the adjacent parts could be connected to the structure at different times during construction. Therefore, if considered necessary, the actual time of connection should be taken into account in the deflection calculation.

7.3.2 Excessive response to wind loads

The response of the structure under wind loads may be assessed using either static or dynamic analysis.

When carrying out *static analysis*, the static characteristic wind loads should be applied to the building and then the lateral deflections at top and every storey of the building evaluated. The lateral deflection at top of the building should not exceed height/500, while the relative lateral deflection in any one storey should not cause damage to the partitions, claddings and finishes etc. Depending on the wind environment, the height/width ratio and the structural efficiency (in terms of lateral load resistance) of the building, this top deflection limit of height/500 may govern the design of a tall building. Hence, for a relatively tall building (more than, say, 40 storeys high), every effort should be made to maximize the lateral stiffness of the building, e.g. by converting the non-structural partition walls into structural walls, aligning the shear walls so that they can be coupled together, increasing the depth of the coupling beams so as to increase the effectiveness of the coupled shear walls, incorporating out-rigger trusses/beams to make good use of the peripheral columns, putting in diagonal bracing members if practicable, and reducing the shear lag of any framed tubes etc. On the other hand, if the relative lateral deflection in any one storey is likely to cause damage to the partitions, claddings and finishes, the mountings of the partitions, claddings and finishes should be detailed to allow for such deformation.

Where a *dynamic analysis* is undertaken, wind speeds based on a 10-year return period of 10 minutes duration should be adopted. The resulting peak acceleration of the building should not exceed 0.15 m/s$^2$ for a residential building or 0.25 m/s$^2$ for an office or a hotel building. If dampers are incorporated to reduce the wind-induced vibration, the design should be supported by dynamic analysis following specialist advice.

It is interesting to compare the above peak acceleration limits of 0.15 m/s$^2$ and 0.25 m/s$^2$ in the code to the corresponding acceleration limits for different comfort factors given in the explanatory notes to the Chinese Code JGJ 3-2002, which are summarized below:

| | Comfort factor | Acceleration limit (in terms of g and m/s$^2$) | |
|---|---|---|---|
| • | No feeling | < 0.005g | (< 0.05 m/s$^2$) |
| • | Feeling | 0.005g ~ 0.015g | (0.05 ~ 0.15 m/s$^2$) |
| • | Disturbing | 0.015g ~ 0.05 g | (0.15 ~ 0.5 m/s$^2$) |
| • | Very disturbing | 0.05g ~ 0.15 g | (0.5 ~ 1.5 m/s$^2$) |
| • | Intolerable | > 0.15 g | (> 1.5 m/s$^2$) |

Hence, the peak acceleration limit of 0.15 m/s$^2$ is approximately the limit that sensitive occupants will feel disturbing, while the peak acceleration limit of 0.25 m/s$^2$ falls well within the range that average occupants will feel disturbing.

The Code of Practice on Wind Effects in Hong Kong 2019 included an acceptable occupant comfort level which is building's frequency dependent. The requirement is reproduced in Figure 7.6. The maximum allowable acceleration basically align with the values stated above.

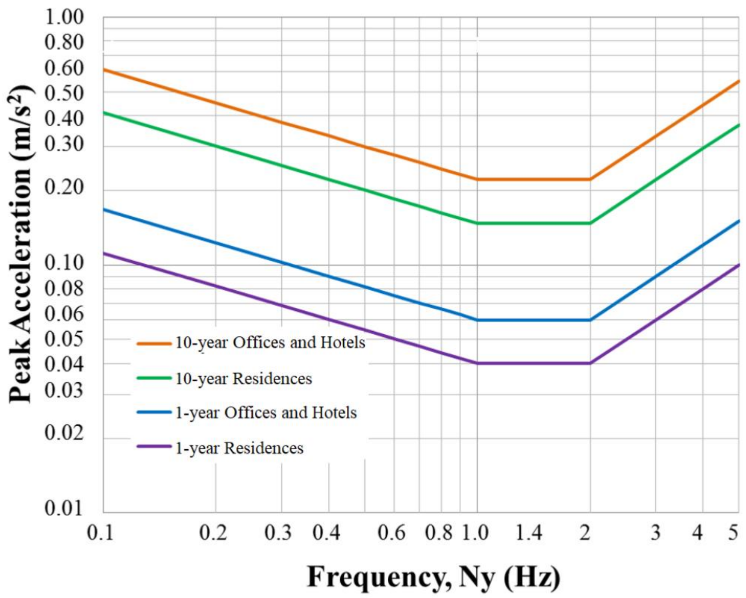



Figure 7.6 Acceptable occupant comfort level
(Adapted from Code of Practice on Wind Effects in Hong Kong 2019)

### 7.3.3 Excessive vibration

Excessive vibration may cause discomfort or alarm to the occupants. To avoid causing discomfort or alarm, long-span floor/footbridge structures should be analysed for their dynamic behaviour under fluctuating loads so as to ascertain their acceptability in terms of vibration control.

If there is any machinery generating vibration that could cause nuisance to the occupants, the machinery should be isolated so that its vibration would not be transmitted to the floor.

If there is any sensitive equipment whose functions could be affected by vibration, it should be easier to isolate the equipment from the floor vibration than to reduce the vibration of the floor.

### 7.3.4 Limiting deflection without direct calculation (deemed-to-satisfy)

In normal circumstances, provided the deemed-to-satisfy limits given in the code with respect to span/effective depth ratios are complied with, no direct calculation of the deflection is required. However, if for some reason, deflection limits more stringent than those stipulated in the code are to be enforced, more rigorous checks would be necessary.

Table 7.3 of the code gives the maximum limits of span/effective depth ratios to be applied to rectangular beams, flanged beams and solid slabs with different support conditions for complying with the permanent sag and deflection after construction requirements in Clause 7.3.1. No allowance has been made for any pre-camber in the derivation of these limits.

The deemed-to-satisfy maximum limits of span/effective depth ratios in Table 7.3 are fairly conservative and may be exceeded if direct calculation of deflection has been carried out to verify compliance with the requirements in Clause 7.3.1.

If it is not necessary to limit the deflection of the structure after construction (i.e. if the deflection of the structure after construction is unlikely to cause any damage), Table 7.3 may be applied to spans longer than 10 m without any modification. But if it is necessary to limit the deflection of the structure after construction, the values in Table 7.3 should be multiplied by the ratio 10/span (the span should be expressed in m) before applying to spans longer than 10 m, except for cantilevers where the design should be justified by direct calculation.

To allow for the effects of the amount of tension reinforcement (which may be estimated from the design ultimate moment) and the service stress (which may be estimated from the ratio of area of tension reinforcement required to area of tension reinforcement provided), the maximum limits of span/effective depth ratios in Table 7.3 of the code should be multiplied by the modification factors given in Table 7.4 of the code. The modification factor varies from 0.76 for a heavily loaded member to 2.0 for a lightly loaded member.

To allow for the effect of the amount of compression reinforcement (expressed in terms of the compression steel area ratio provided), the maximum limits of span/effective depth ratios in Table 7.3, after being multiplied by the modification factors given in Table 7.4, should be further multiplied by the modification factors given in Table 7.5.

The code suggests that if creep or shrinkage of the concrete could be particularly high or if other abnormally adverse conditions are expected, the permissible span/effective depth ratios should be suitably reduced. However, it is not mentioned how the reductions in permissible span/effective depth ratios should be determined. The author would like to propose that, in such case, the deflection control should be exercised by direct calculation of the deflection of the structure, taking into account the creep/shrinkage of the concrete and any abnormally adverse conditions, and checking the deflection results against the permissible deflection limits. Unfortunately, the shrinkage of concrete in Hong Kong is quite high and it seems that we cannot avoid this problem.

There is actually another problem that may be more serious. The elastic modulus of the concrete in Hong Kong is relatively low and thus the maximum span/effective depth ratios given in Table 7.3 of the code, which are based on BS8110:

Part 1: 1997, may have to be adjusted to reflect the difference in elastic modulus. Further research on this topic is needed.

### 7.3.5 Calculation of deflection

For deflection control by direct calculation, only the long-term deflections (the total deflection and the deflection after construction) under the quasi-permanent load combination need to be evaluated and checked. The quasi-permanent load combination comprises of the dead load and the quasi-permanent values of the imposed loads. Since the dead load is known to within close limits, lack of knowledge of the precise quasi-permanent values of the imposed loads is not likely to be a major source of error in the deflection calculation. The main difficulties are with estimation of the rotational restraint/stiffness of the supports, realistic structural modelling of the non-structures, allowing for creep effects and allowing for tension stiffening of cracked concrete etc.

The deflection of a member subjected to flexure may be calculated from the curvature using the following equation:

$$\frac{\partial^2 a}{\partial x^2} = \frac{1}{r_x} \qquad \text{Equation 7.4}$$

where $a$ is the deflection at $x$ and $1/r_x$ is the curvature at $x$. The deflection $a$ may be obtained by calculating the curvature $1/r_x$ at successive sections along the member and solving the above equation by numerical integration. Alternatively, the following approximate formula may be used:

$$a = K\frac{l^2}{r_b} \qquad \text{Equation 7.5}$$

where $l$ is the effective span of the member, $1/r_b$ is the curvature at mid-span or for cantilevers at the support section, and $K$ is a coefficient depending on the shape of the bending moment diagram. The $K$ values are given in Table 7.6 of the code. It should be noted that the $K$ values for cantilever spans are generally much larger than those for simple and continuous spans.

The above methods are applicable only to relatively simple cases. In more complicated cases, the finite element method may have to be used.

### 7.3.6 Calculation of curvature

The curvature of a section may be calculated based on the following assumptions:
*Cracked section:*

- plane sections remain plane;
- the reinforcement is elastic with an elastic modulus of 200 kN/mm$^2$;
- the concrete in compression is elastic;
- the tensile stress developed in the concrete varies from 0 N/mm$^2$ at the neutral axis to 1.0 N/mm$^2$ instantaneously or 0.55 N/mm$^2$ in the long-term at the level of the tension reinforcement within the tension zone.

*Uncracked section:*

- plane sections remain plane;
- the reinforcement is elastic with an elastic modulus of 200 kN/mm$^2$;
- the concrete in compression and in tension is elastic;
- the maximum tensile stress developed in the concrete is not larger than 1.0 N/mm$^2$.

In assessing the long-term curvature, the creep effect has to be taken into account by using the long-term elastic modulus of concrete in the analysis. The long-term elastic modulus may be taken as $1/(1+\phi)$ times the short-term elastic modulus, where $\phi$ is the appropriate creep coefficient.

The shrinkage curvature of the concrete should be added to the curvature due to dead and imposed load. It may be calculated using the following equation:

$$\frac{1}{r_{cs}} = \frac{\rho_0 \varepsilon_{cs}}{d} \qquad \text{Equation 7.7}$$

where $1/r_{cs}$ is the shrinkage curvature, $\varepsilon_{cs}$ is the shrinkage strain, $\rho_0$ is a coefficient depending on the tension and compression steel ratios and $d$ is the effective depth of the section. The values of $\rho_0$ are given in Table 7.7 of the code. It is noteworthy that the addition of more tension reinforcement might not help to reduce the shrinkage curvature but increasing the effective depth would.

From Figure 7.2 of the code, it can be seen that the total long-term curvature consists of three components:

(1) the long-term curvature due to the permanent load;
(2) the shrinkage curvature; and
(3) the instantaneous curvature due to increase of permanent load to total load.

# 8 REINFORCEMENT: GENERAL REQUIREMENTS

## 8.1 General

The reinforcement detailing rules given herein are applicable only to normal steel reinforcement, welded fabric and prestressing tendons. They are not applicable to painted, epoxy or zinc coated bars.

The detailing rules are also applicable only to normal building structures subjected predominantly to static loading. They are not applicable to structures/ elements subjected to seismic loading, impact loading, machine vibration and fatigue.

For reinforcing bars to be fitted within two concrete faces, the overall dimension on the bending schedule should be determined as the nominal dimension of the concrete minus the nominal cover on each face minus the appropriate deduction given in Table 8.1 of the code.

## 8.2 Spacing of reinforcement

The spacing and arrangement of the bars should be such that the concrete can be placed and compacted properly and the bars are adequately bonded or anchored.

The clear horizontal distance between individual parallel bars and the clear vertical distance between horizontal layers of parallel bars should be not less than the maximum of the following:

- bar diameter;
- maximum size of aggregate + 5 mm;
- 20 mm.

Bars positioned in separate horizontal layers should be located vertically above each other so as to allow access for vibrators to compact the concrete underneath the bars.

## 8.3 Permissible internal radii for bent bars

The minimum internal radii for bent bars should be such that:

- the bars would not fail by having bending cracks;
- the concrete inside the bends would not fail because of high bearing stress acting from the bars.

To avoid cracking failure of the bars, the minimum internal bend radii should be as given in Table 8.2 of the code (i.e., $2\phi$ when $\phi \leq 12$mm, $3\phi$ when $\phi < 20$mm and $4\phi$ when $\phi \geq 20$mm, in which $\phi$ is the bar diameter). In no case should the minimum internal bend radius be less than twice the internal radius of the test bend guaranteed by the manufacturer. The cross bar shall extend for at least $3\phi$ length on both side of the bend.

If the minimum internal bend radius is not less than the respective limiting value given in Table 8.2 of the code and one of the following conditions is satisfied, the design bearing stress inside the bends needs not be checked:

- the anchorage of the bar does not require a length more than $4\phi$ past the end of the bend;
- the bar is assumed not to be stressed beyond a point $4\phi$ past the end of the bend at ultimate limit state; or
- there is a cross bar of diameter $\geq \phi$ inside the bend.

Otherwise, the design bearing stress inside the bends should be checked to avoid concrete failure. The design bearing stress should be checked using the following equation:

$$\text{bearing stress} = \frac{F_{bt}}{r\emptyset} \leq \frac{2f_{cu}}{\left(1+2\frac{\emptyset}{a_b}\right)} \qquad \text{Equation 8.1}$$

where $F_{bt}$ is the tensile force from ultimate loads at the start of the bend, $r$ is the internal radius and $a_b$ is the centre-to-centre distance between the bars or the cover plus bar diameter, whichever is the smaller.

## 8.4 Anchorage of longitudinal reinforcement

All longitudinal reinforcing bars, wires or welded fabrics should be properly anchored at each of their ends to transmit their forces to the concrete without causing longitudinal splitting or spalling. Longitudinal splitting or spalling may occur because the bond stress developed at the bar/concrete interface could induce bursting pressure onto the surrounding concrete thereby subjecting the concrete to circumferential tension. If necessary, transverse reinforcement should be provided to avoid such kind of failure.

At any section, each bar should be properly anchored at both sides of the section by an appropriate anchorage length or other end anchorage. Provided this is done, local bond stress needs not be checked.

The anchorage bond stress $f_b$ may be assumed to be uniformly distributed over the effective anchorage length and taken as the force in the bar divided by the effective anchorage area, as given by the following equation:

$$f_b = \frac{F_s}{\pi\emptyset l_b} \qquad \text{Equation 8.2}$$

where $F_s$ is the force in the bar and $l_b$ is the effective anchorage length.

The anchorage bond stress $f_b$ should not exceed the design ultimate bond stress $f_{bu}$ given by:

$$f_{bu} = \beta\sqrt{f_{cu}} \qquad \text{Equation 8.3}$$

where $\beta$ is a coefficient depending on bar type and $f_{cu}$ is limited to 60 N/mm$^2$.

For bars in beams where minimum links have been provided in accordance with Table 6.2 of the code, the values of $\beta$ may be taken from Table 8.3 of the code. For bars in beams where minimum links have not been provided in accordance with Table 6.2 of the code, the values of $\beta$ should be taken as those appropriate to plain bars irrespective of the bar type. For bars in slabs, the values of $\beta$ may be taken from Table 8.3 of the code. It should be noted that although the code says that Table 8.3 is for bars in tension, the table is actually applicable to both bars in tension and bars in compression.

The required anchorage length may be calculated using the following equation:

$$l_b = \frac{f_s \emptyset}{4 f_{bu}} \qquad \text{Equation 8.4}$$

where $f_s$ is the ultimate stress developed in the bar taken as $0.87 f_y$. Substituting the values of $f_s$ and $f_{bu}$ into the equation, the required anchorage lengths, expressed as multiples of bar diameter, may be obtained as tabulated in Table 8.4 of the code. Note that although the equations used are the same as those in BS8110: Part 1: 1997, since the material safety factor of steel is 1.15 in the new code but is 1.05 in BS8110, the required anchorage lengths in the new code are not the same as those in BS8110. As a rule of thumb, an anchorage length of 40 times bar diameter should be sufficient in most cases.

Bearing failure of concrete inside the bends should be prevented by ensuring that the minimum internal bend radius complies with the requirement in Table 8.2 and that either the anchorage does not require a length more than $4\phi$ past the end of the bend or the bearing stress inside the bend satisfies the requirement of Equation 8.1.

Regarding the effective anchorage length of a bend or a hook, both BS8110 and Eurocode 2 have provided some guidelines. Herein, it is suggested to follow BS8110. According to Clause 3.12.8.23 of BS8110: Part 1: 1997, the effective anchorage length of a bend or hook is the length of a straight bar which would be equivalent in terms of anchorage to that portion of the bar between the start of the bend and a point 4 times the bar diameter beyond the end of the bend. This effective anchorage length may be taken as:

For a *90$^o$ bend:* either 4 times the internal radius of the bend with a maximum of 12 times the bar diameter, or the actual length of the bar, whichever is greater.

For a *180$^o$ hook:* either 8 times the internal radius of the hook with a maximum of 24 times the bar diameter, or the actual length of the bar, whichever is greater.

If the internal radius is at least 3 times the bar diameter, then the effective anchorage length should be the maximum of (12 times the bar diameter or actual length of bar) for a 90$^o$ bend or the maximum of (24 times the bar diameter or actual length of bar) for a 180$^o$ hook.

For bottom bars in simply supported beams, no bend or hook should begin before the centre of the support or before half of the effective depth from the face of the support (see Clause 9.2.1.7 of the code).

## 8.5 Anchorage of links and shear reinforcement

Each end of a link or shear reinforcement should be anchored by one of the following means:

- passing round a longitudinal bar of at least the same bar size to form a bend or hook and continuing for a certain minimum length beyond the bend, or
- welding to it at least one or two perpendicular bars (the perpendicular bars may be longitudinal or transverse).

The detailed requirements for the bends/hooks or welded bar anchorages are shown in Figure 8.2 of the code.

## 8.6 Anchorage by welded bars

Additional anchorage may be provided by welding transverse bars (also called cross bars) bearing on the concrete, as shown in Figure 8.3 of the code.

If there is one such welded transverse bar, the anchorage capacity $F_{\mathrm{btd}}$ may be calculated as:

$$F_{\mathrm{btd}} = l_{td}\phi_t\sigma_{td}, \text{ but not greater than } F_{\mathrm{wd}} \qquad \text{Equation 8.7}$$

in which $l_{td}$ is the design length of transverse bar, $\phi_t$ is the diameter of transverse bar, $\sigma_{td}$ is the concrete stress and $F_{\mathrm{wd}}$ is the design shear strength of weld.

If there are two such welded transverse bars, one on each side of the bar to be anchored, the capacity given by the above equation should be doubled.

If there are two such welded transverse bars, on the same side of the bar to be anchored with a minimum spacing of 3 times the bar size, the capacity should be multiplied by a factor of 1.4.

## 8.7 Laps

Axial force may be transmitted from one bar to the next in the same direction by:

- lapping of bars;
- welding (not for joints subjected to cyclic loading to avoid fatigue failure);
- using a mechanical coupler.

The requirements for mechanical couplers have already been stipulated in Section 3.2. Hence, only the requirements for laps are given in this section. Most of the requirements for laps in the new code are adopted from Eurocode 2 and BS8110: Part 1: 1997.

The detailing of laps between bars should be such that:

- transmission of forces from one bar to the next is assured;
- spalling of concrete cover near the lap does not occur; and

- longitudinal cracks along the bars do not occur.

Laps between bars should not be located in areas of large moments/forces or expected plastic hinge locations and should preferably be staggered. As a general principle, laps in any section should be arranged symmetrically. Moreover, to ensure proper consolidation of concrete, the sum of reinforcement sizes in a particular layer should not exceed 40% of the breadth of the section at that level.

Main bars in compression and secondary (distribution) bars in tension or compression may be lapped in one section. However, main bars in tension must not be lapped in one section unless the requirements stipulated in Figure 8.4 of the code are complied with.

The requirements stipulated in Figure 8.4 of the code are:

- clear transverse distance between two lapping bars should not be greater than $4\phi$ or 50 mm; otherwise, the lap length should be increased;
- longitudinal distance between adjacent laps should not be less than 0.3 times the lap length; and
- clear distance between adjacent bars in adjacent laps should not be less than $2\phi$ or 20 mm.

If these requirements are met, all the main bars in tension and in one layer may be lapped in one section. However, despite satisfaction of these requirements, main bars in tension and in several layers should have at most 50% of them lapping at any one section.

*Minimum lap length:*
The minimum lap length should be:

- for bar reinforcement: not less than $15\phi$ or 300 mm;
- for fabric reinforcement: not less than 250 mm.

*Tension lap length:*
The tension lap length should be at least equal to the design tension anchorage length. If the lapped bars are of unequal size, the lap length may be based on the smaller bar. Under the following conditions, the tension lap length needs to be increased (an illustration has been given in Figure 8.5 of the code):

(a) if the lap is at top of a section as cast and the minimum cover is less than $2\phi$, the lap length should be increased to 1.4 times the anchorage length;
(b) if the lap is at a corner of a section as cast and the minimum cover to either face is less than $2\phi$, the lap length should be increased to 1.4 times the anchorage length;
(c) if the clear distance between adjacent laps is less than $6\phi$ or 75 mm, the lap length should be increased to 1.4 times the anchorage length;
(d) if both conditions (a) and (b) apply, the lap length should be increased to 2.0 times the anchorage length;
(e) if both conditions (a) and (c) apply, the lap length should be increased to 2.0 times the anchorage length.

There are three basic reasons why the lap length has to be increased for laps at top, laps at corners, laps with relatively small cover and laps with relatively small clear distance from each other. Firstly, concrete as cast is not entirely uniform. When a fresh concrete is placed, the solid particles in the concrete mix will tend to sink because of their heavier weight compared to water (this downward movement of the solid particles is called sedimentation). Due to sedimentation of the solid particles, the water in the concrete mix will rise to the top (this upward movement of the water is called bleeding). As a result, the concrete near the top has a higher water/cementitious ratio and a lower strength whereas the concrete near the bottom has a lower water/cementitious ratio and a higher strength. Moreover, near the top of a section, the concrete underneath fixed reinforcing bars tends to be less densely compacted (mainly because of sedimentation) and generally contains more bleeding water (because uprising water has been trapped there). Due to both higher water/cementitious ratio at the top and lower quality of concrete underneath top reinforcing bars, the bond strength of concrete is significantly lower near the top. Secondly, the bond strength is dependent on how well the reinforcing bars are embedded. If the bars are not sufficiently well embedded, e.g. if the bars are located at corners or the bars are not provided with generous cover, the bond strength tends to be weaker. Thirdly, a close spacing of adjacent laps could lead to spalling or longitudinal cracking of the concrete cover, which would then weaken the bond between the concrete and the reinforcing bars.

*Compression lap length:*
The compression lap length should be at least equal to 1.25 times the design compression anchorage length. If the lapped bars are of unequal size, the lap length may be based on the smaller bar.

*Transverse reinforcement:*
Transverse reinforcement is needed for both tension and compression laps to resist the transverse tension forces, which tend to cause spalling or longitudinal cracking of the concrete. Where $\phi$ is less than 20 mm or the percentage of lapped bars is less than 25%, any transverse reinforcement provided for other purposes may be assumed sufficient for the transverse tension forces without further justification. Otherwise, transverse reinforcement at least equal to the area of one lapped bar should be added. If more than 50% of the bars are lapped at one section and the distance between adjacent laps is not greater than $10\phi$, the transverse reinforcement should be formed by links or U-bars anchored into the body of the section. In compression laps, one additional transverse bar should be placed beyond each end of the lap length and within $4\phi$ of the end of the lap length to resist the burst forces induced by end bearing of the compression bars. Any transverse reinforcement provided should be positioned at the outside of the laps.

## 8.8 Additional rules for large diameter bars

The definition of large diameter bars may vary from place to place (Eurocode 2 allows different definitions to be adopted in different countries). In the new code, bars of diameter larger than 40 mm are regarded as large diameter bars. For such large diameter bars, the additional rules in Section 8.8 of the code would apply. These additional rules are necessary because with the use of large diameter bars, the splitting and dowel forces are greater and crack control would be more critical and difficult.

When large diameter bars are used, crack control should be achieved either by using surface reinforcement or by direct calculation (as per Clause 7.2.3 of the code). The code refers to Clause 9.2.4 for the design of the surface reinforcement but there is no Clause 9.2.4 in the code. The code is actually referring to Clause 9.2.4 of Eurocode 2. According to Clause 9.2.4 and Annex J of Eurocode 2, surface reinforcement should be provided to resist spalling wherever the main reinforcement is made up of large diameter bars. If provided, the surface reinforcement should consist of wire mesh or small diameter bars and be placed outside the links. The area of surface reinforcement should not be less than $0.01A_{\mathrm{ct,ext}}$ in the direction perpendicular to the large diameter bars and $0.02A_{\mathrm{ct,ext}}$ parallel to those bars.

Large diameter bars should be anchored with mechanical devices. As an alternative, they may be anchored in the form of straight bars, but links should be provided as confining reinforcement to prevent splitting.

In the anchorage zones of the large diameter bars, if transverse compression is not present, transverse reinforcement in the form of links, additional to that provided for other purposes, should be provided. For straight anchorage lengths, the additional transverse reinforcement should not be less than the following:

- in the direction parallel to the tension face, $A_{sh} = 0.25A_s n_1$
- in the direction perpendicular to the tension face, $A_{sv} = 0.25A_s n_2$

The additional transverse reinforcement bars should be uniformly distributed in the anchorage zone and their spacing should not exceed 5 times the diameter of the large diameter bars.

Generally, large diameter bars should not be lapped, except in sections with a minimum dimension of 1.0 m or where the stresses in the bars are not greater than 80% of the design ultimate strength. For large diameter bars, the use of mechanical couplers should be preferred.

## 8.9 Bundled bars

In a bundle, all the bars should be of the same type and grade. The bars may be of different sizes provided the ratio of diameters does not exceed 1.7.

A bundle of bars may be treated as equivalent to a notional bar having the same sectional area. The number of bars in the bundle is limited to a maximum of 4 for vertical bars in compression and bars in a lapping joint or to a maximum of 3 for all other cases. Moreover, the equivalent diameter is limited to not greater than 55 mm.

*Tension anchorages of bundled bars:*
For tension anchorages, a bundle of bars with an equivalent diameter smaller than 32 mm may be curtailed near a support without staggering (in this case, the required anchorage length should be based on the equivalent diameter of the bundle). However, a bundle of bars with an equivalent diameter greater than or equal to 32

mm, which are to be anchored near a support, needs to be curtailed in a staggered manner as shown in Figure 8.8 of the code. Basically, the second bar should be curtailed at a distance of at least $1.3l_b$ from the end of the first bar and the third bar should be curtailed at a distance of at least $l_b$ from the end of the second bar (in this case, the value of $l_b$ should be based on the diameter of an individual bar).

*Compression anchorages of bundled bars:*
For compression anchorages, a bundle of bars, regardless of its equivalent diameter, may be curtailed without staggering. For a bundle of bars with an equivalent diameter greater than or equal to 32 mm, at least four links having a diameter of not less than 12 mm should be provided at each end of the bundle and a further link should be provided just beyond the end of the bundle. The five links are to serve as confining reinforcement to prevent splitting.

*Lapping of bundled bars:*
For a bundle consisting of not more than two bars and with an equivalent diameter less than 32 mm, the bundle of bars may be lapped without staggering (in this case, the equivalent diameter should be used to calculate the lap length). Otherwise, the lapping should be staggered in the longitudinal direction by at least $1.3l_0$ (in this case, the value of $l_0$ should be based on the diameter of an individual bar). A fourth bar of the same size may be used to form the staggered lap without cranking the bars or shifting the centroidal axis of the bundle, as shown in Figure 8.9 of the code. At any lap location, there should not be more than four bars. For this reason, a bundle of more than three bars should not be lapped.

## 8.10 Prestressing tendons

*Layout of pre-tensioned tendons:*
The minimum clear horizontal spacing of pre-tensioned tendons should be not less than the maximum of the following:

- tendon diameter;
- maximum size of aggregate + 5 mm;
- 20 mm.

The minimum clear vertical spacing of pre-tensioned tendons should be not less than the maximum of the following:

- tendon diameter;
- maximum size of aggregate.

Other layouts may be used provided that test results have shown satisfactory ultimate behaviour with respect to proper anchorage of the tendons, concrete compression at the anchorages, spalling of concrete and placing of concrete between the tendons. Bundling of tendons should not be allowed within the anchorage zones, unless it has been shown that there are no problems with placing and compaction of concrete and bonding of the tendons. It should be noted that the above minimum clear spacing requirements are not as stringent as those stipulated in Eurocode 2 and therefore care should be exercised to check during field applications whether the above requirements are sufficient to ensure satisfactory performance.

*Layout of post-tensioned tendons:*
The minimum clear horizontal spacing of post-tensioned tendons should be not less than the maximum of the following:

- duct diameter;
- maximum size of aggregate + 5 mm;
- 50 mm.

The minimum clear vertical spacing of post-tensioned tendons should be not less than the maximum of the following:

- duct diameter;
- maximum size of aggregate;
- 40 mm.

Bundling of ducts should not be allowed except in the case of a pair of ducts placed vertically one above the other. Where curved tendons are used, appropriate measures (such as positioning of the tendon ducts and provision of bursting and restraining reinforcement) should be taken so as to prevent:

- bursting of the side cover perpendicular to the plane of curvature;
- bursting of the cover in the plane of curvature;
- crushing of the concrete separating ducts in the same plane of curvature.

In order to prevent bursting of the side cover perpendicular to the plane of curvature, the cover should be in accordance with the values given in Table 8.6 of the code. In order to prevent bursting of the cover in the plane of curvature, which tends to occur at locations where the curved tendons run close to an external surface of concrete and develop radial forces acting outwards, restraining reinforcement should be provided to tie the tendons back into the core of the concrete member. In order to prevent crushing of the concrete between ducts, the above minimum clear spacing requirements should be met and the minimum spacing in the plane of curvature should be in accordance with the values given in Table 8.7 of the code.

*Anchorages of pre-tensioned tendons:*
The design of the anchorages should be such that:

- sufficient transmission lengths are developed to avoid bond-slip failure; and
- longitudinal splitting of concrete due to stress concentration is avoided.

The transmission length of a tendon in concrete is dependent on:

- type, size and surface condition of the tendon;
- degree of compaction and strength of the concrete; and
- location of tendon.

Transmission lengths for tendons near the top are generally greater than those for identical tendons near the bottom. This is due to sedimentation and bleeding of the concrete mix during the plastic stage (sedimentation causes the concrete underneath the top tendons to be less densely compacted while bleeding causes the concrete near the top to have a higher water/cementitious ratio), as in the case of ordinary reinforcing bars in concrete. It should be noted that the transmission length can vary a great deal for different factory or site conditions. For example, it has been shown that the transmission length for wires and strands may vary between 100 and 160 diameters. As far as possible, therefore, the transmission length assumed in the design should be based on experimental evidence for the known factory or site conditions. In the absence of test data, the following equation may be used to estimate the transmission length $l_t$ for initial prestressing forces up to 75% of the characteristic strength of the tendons:

$$l_t = \frac{K_t \phi}{\sqrt{f_{ci}}} \qquad \text{Equation 8.10}$$

in which $f_{ci}$ is the concrete strength at transfer, $\phi$ is the nominal diameter of the tendon and $K_t$ is a coefficient depending on the type of tendon. From the values presented in the code, it can be seen that $K_t$ varies from 240 for 7-wire standard or super strand to 600 for plain or indented wire.

*Anchorages/couplers/deviators of post-tensioned tendons:*
The design of each anchorage should take into account:

- bearing stress behind the anchorage plate;
- bursting forces around the anchorage;
- spalling of concrete around the anchorage; and
- overall equilibrium of the anchor block.

The bearing capacity of concrete is dependent on how localized the bearing stress is and the amount of confinement provided. The code has not provided any guideline for checking the bearing stresses behind the anchorage plates; specialist literature should be consulted for such purpose. On the other hand, the bursting forces should be evaluated using an appropriate strut and tie model. For such kind of modelling, readers should refer to Clause 6.5 of Eurocode 2 (the code refers to Clause 6.5 of the code, which has nothing to do with strut and tie modelling). As a simplification, the concentrated prestressing force may be assumed to disperse at an angle of spread of $2\beta$ starting at the end of the anchorage device, where $\beta$ may be assumed to be $33.7^{o}$. Where couplers are used, they should be designed in accordance with the specification of the prestressing system and should be so placed that they would not affect the bearing capacity of the member and that any temporary anchorage needed during construction could be introduced in a satisfactory manner. In general, couplers should be located away from supports. The placing of couplers for 50% or more of the tendons at one section should be avoided unless it can be shown that the temporary anchorage of such a high proportion of the total tendon forces at one section would not cause any safety problem. Where deviators are provided, each should be designed to satisfy the following requirements:

- the deviator should be able to withstand both longitudinal and transverse forces that the tendon applies to it and transmit these forces to the structure; and
- the deviator should have a suitable radius of curvature to ensure that the tendon within the deviator would not cause any overstressing or damage to it.

In the deviator zones, the tubes forming the sheaths should be able to sustain the radial pressure and longitudinal movement of the tendons, without causing any damage and without impairing its proper function. The forces developed by the change of angle using a deviator should be taken into account in the design calculations. If the angle of deviation is less than 0.01 radian, a deviator may not be necessary.

# 9 DETAILING OF MEMBERS AND PARTICULAR RULES

## 9.1 General

The detailing requirements in this chapter supplement those in Chapter 8 and have to be complied with at the same time.

## 9.2 Beams

### 9.2.1 Longitudinal reinforcement

The minimum percentages of longitudinal reinforcement to be provided are given in Table 9.1. A new factor of $\alpha_{min}$ is involved. These minimum percentages are based on BS8110: Part 1: 1997. The basic principle for the provision of minimum tension reinforcement is to ensure that the cracked concrete section would have at least the same tensile or flexural strength as the uncracked concrete section so that when concrete cracks, the section would not fail immediately. The basic principle for the provision of minimum compression reinforcement is to ensure the attainment of a certain level of ductility when the surrounding concrete fails in compression. The basic principle for the provision of minimum transverse reinforcement in flanges is to cater for the bursting forces when the web horizontal shear forces transmitted to the flanges are dispersed into the flanges.

To avoid congestion, neither the area of tension reinforcement nor the area of compression reinforcement should exceed 4%. At laps, the sum of the bar sizes in a particular layer should not exceed 40% of the breadth of the section.

For the purpose of crack control, longitudinal bars should be added to the side faces of beams exceeding 750 mm overall depth. The minimum size of the longitudinal bars in the side faces should be $\sqrt{\left(s_b b / f_y\right)}$ where $s_b$ is the bar spacing and $b$ is the breadth of the section or 500 mm if $b > 500\text{mm}$. The bar spacing of such longitudinal bars should not exceed 250 mm.

Also for the purpose of crack control, the clear distance between bars or groups of bars near the tension face should be limited by:

$$\text{clear spacing} \leq \frac{70000\beta_b}{f_y} \leq 300 \text{ mm} \qquad \text{Equation 9.1}$$

where $\beta_b$ is the moment redistribution ratio. Alternatively, the clear spacing may be assessed by:

$$\text{clear spacing} \leq \frac{47000}{f_s} \leq 300 \text{ mm} \qquad \text{Equation 9.2}$$

in which $f_s$ is the estimated service stress in the reinforcement. The distance between the face of the beam and the nearest longitudinal bar in tension should not be greater than half of the clear distance determined above.

The code suggests that in monolithic construction, even when simple supports have been assumed in the design, the sections at supports should be designed for a hogging moment arising from partial fixity of at least 15% of the maximum sagging moment in the span. However, the author does not recommend the practice of assuming monolithic joints as simple supports because such practice, even with the joints designed for 15% of the maximum bending moment in the span, could lead to cracking of the joints. Monolithic joints should be designed as monolithic joints with perhaps some redistribution of moment applied after obtaining the moment diagram. Simplification just for the purpose of saving time in the design calculations is not a good practice. The performance of the finished structure should be the primary consideration.

At intermediate supports of continuous beams, the tension reinforcement in the flange of a flanged section may be spread over the effective width of the flange provided that at least 50% of the total area of tension reinforcement is placed within the web width.

Except at end supports, in every flexural member, every bar should extend beyond the point at which it is no longer needed (the point at which it is no longer needed is the point where the design resistance moment of the section, considering only the continuing bars, is equal to the design moment at ultimate limit state) for a distance of at least the effective depth of the member or 12 times the bar diameter, whichever is larger. In addition, every bar in tension should extend:

- beyond the point at which it is no longer needed to assist in resisting bending moment for a distance equal to the anchorage length appropriate to its design strength ($0.87 f_y$); or
- to the point where the design shear capacity of the section is greater than twice the design shear force at that section; or
- to the point where other bars continuing past that point provide double the area required to resist the design bending moment at that section.

Curtailment of a substantial amount of reinforcement at any section could lead to the development of large longitudinal and transverse cracks and should be avoided. It is therefore advisable to stagger the curtailment points in heavily reinforced members.

At an end support, which acts as a simple support, at least 50% of the calculated mid-span bottom reinforcement should be extended to the end support and anchored there by one of the following:

- an effective anchorage length of 12 times the bar diameter beyond the centre of the support (no bend or hook should begin before the centre of the support), or
- an effective anchorage length of 12 times the bar diameter beyond a distance of half the effective depth from the face of the support (no bend or hook should begin before half of the effective depth from the face of the support).

At an intermediate support of a continuous beam, at least 30% of the calculated mid-span bottom reinforcement should be continuous through the support.

Where a continuous beam or slab is extended beyond the end support to form a cantilever extension, the top reinforcement of the cantilever extension should be extended back to beyond the point of contraflexure in the adjacent span.

All compression reinforcement in beams should be properly contained by links passing round each corner bar and each alternate bar in an outer layer having an included angle of 135° or less, as for the case of compression reinforcement in columns (see Clause 9.5.2.2 of the code).

### 9.2.2 Shear reinforcement

The shear reinforcement may consist of a combination of:

- links enclosing the longitudinal tension bars and the compression zone;
- bent-up bars; and
- cages or ladders not enclosing the longitudinal bars but are properly anchored at each end in the compression and tension zones.

At least 50% of the necessary shear reinforcement should be in the form of links.

The minimum links to be provided should be in accordance with Table 6.2 of the code, i.e.:

$$A_{sv} \geq {v_r b_v s_v}/{0.87 f_{yv}} \qquad \text{Equation 9.4}$$

in which $v_r$ is the nominal shear strength to be provided. For $f_{cu} \leq 40\,\mathrm{N/mm^2}$, $v_r = 0.4\,\mathrm{N/mm^2}$ while for $f_{cu} > 40\,\mathrm{N/mm^2}$, $v_r = 0.4\left({f_{cu}}/{40}\right)^{2/3}$ with $f_{cu}$ taken as $80\,\mathrm{N/mm^2}$ when it is higher than $80\,\mathrm{N/mm^2}$ (note that the formula given in Clause 9.2.2 of the code is applicable only when $f_{cu} \leq 40\,\mathrm{N/mm^2}$).

In the direction of the span, the maximum spacing of the links should not exceed 0.75 times the effective depth. In the direction perpendicular to the span, every longitudinal tension bar should be within 150 mm from a vertical leg of a link, a cage or a ladder. The truss analogy would suggest a maximum spacing equal to the lever arm: 0.75*d* is a lower bound value for this. The logic behind the limit on lateral spacing of the legs of stirrups is less clear but experimental evidence suggests a reason why such a limit is valuable. One of the major functions of stirrups is to inhibit "dowel" failure of the tension steel. This is a tearing out of a bar in the way sketched in Figure 9.1. The effectiveness of the stirrup in achieving this will reduce with increasing distance of the vertical leg. If a bar is placed further than 150mm from a stirrup leg, it can still be used to provide flexural strength but may be ignored in assessing $v_c$.

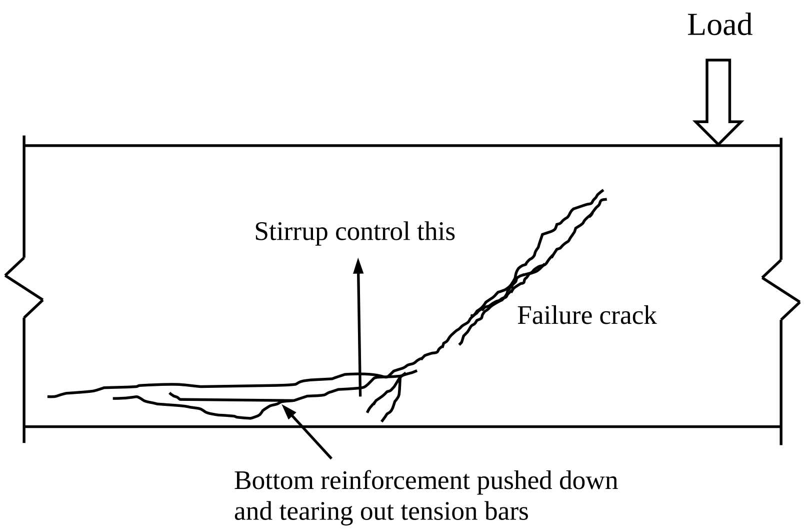



Figure 9.1: Normal mode of shear failure

(Adapted from Rowe, R.E., Handbook to British Standard BS 8110:1995 – Structural Use of Concrete, London, England: Palladian Publications, 1987, 206pp)

9.2.3 Torsion reinforcement

The torsion reinforcement should consist of rectangular closed links together with longitudinal reinforcement and is additional to any requirements for shear or bending.

The torsion links should be closed by laps or anchored by 135$^o$ hooks. Some recommended shapes are shown in Figure 9.2. They should be placed in a plane perpendicular to the axis of the member. Their spacing should not exceed the minimum of (the smaller dimension of the link, half of the larger dimension of the link or 200 mm). On the other hand, the longitudinal reinforcement should be distributed evenly round the inside perimeter of the links with a clear distance between them of not exceeding 300 mm and at least four bars, one at each corner should be provided.

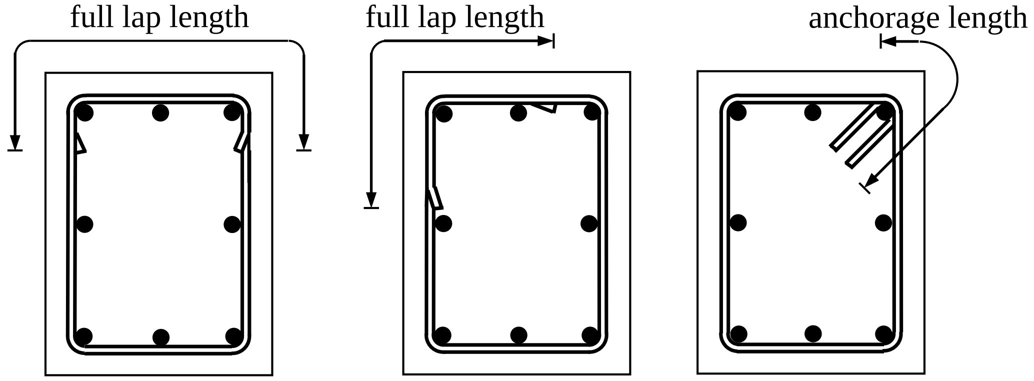



Figure 9.2 Some torsion link arrangements

The importance of the good anchorage of the transverse reinforcements in resisting torsion has been diagrammatically explained by Law & Mak (2013) as extracted.

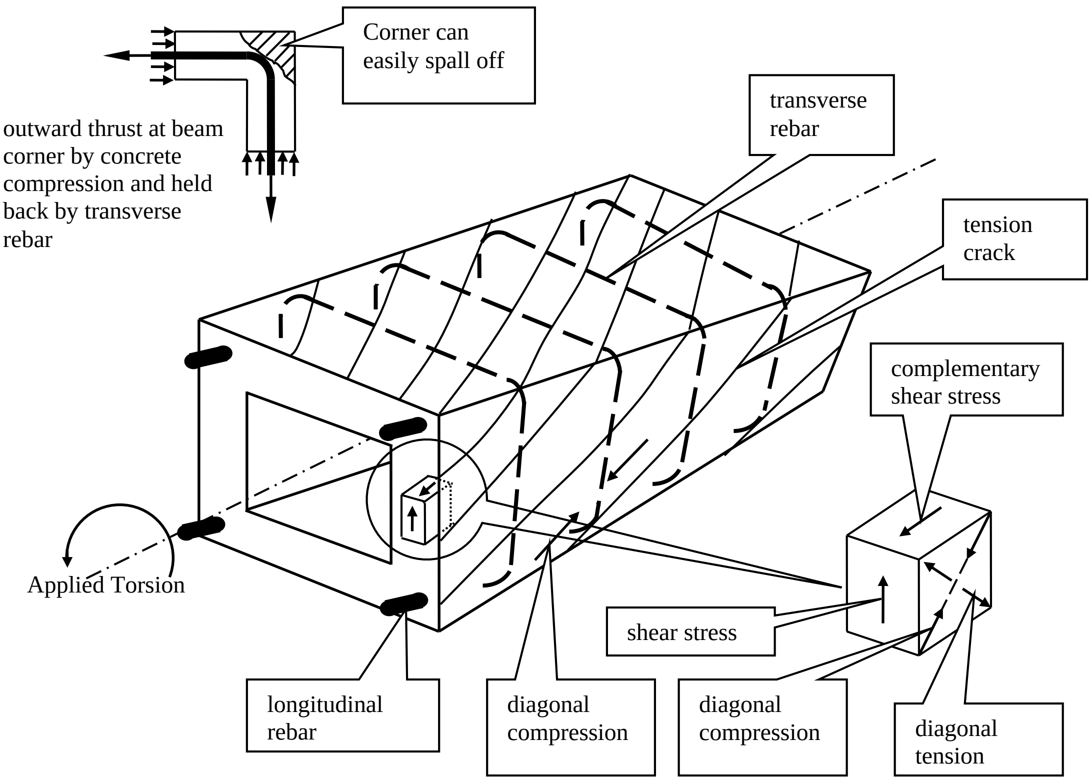



Figure 9.3 The Diagonal Compression Stress Field Model for Torsion

(Adapted from Law C.W., Mak J.Y.W., "Some explanatory/commentary notes on reinforced concrete detailing in accordance with the Code of Practice for Structural Use of Concrete 2004", The HKIE Transactions Vol. 20 No. 1, 2013, pp34-51)

From the Figure 9.3, in which a beam is under torsion, its internal stresses will comprise complementary shear stresses, diagonal tension and compression as illustrated by the cubical element in the diagram. In the ultimate limit state, the diagonal tensions will crack the concrete and the diagonal compression which spiral around the beam will take place to achieve equilibrium. In addition, the diagonal compression will create outward thrust at the corner and the concrete outside the transverse reinforcement may easily "spall" off as shown in the diagram. In fact, the compression will create outward thrusts on the whole beam and the longitudinal and transverse reinforcements are required to tie the beam so as to prevent it from splitting. Adequate anchorages of both the longitudinal and transverse reinforcements are therefore essential. That is the reason why the transverse reinforcements need to be well anchored by adequate laps. Law & Mak (2013) further illustrated with diagram for the failure due to improper anchorage of the transverse reinforcements as below :

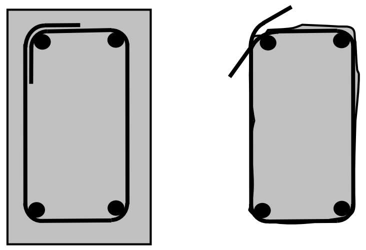



Figure 9.4(a) – Loss of Anchorage of Stirrups formed by 90° Bend End

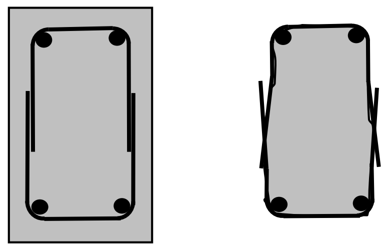



Figure 9.4(b) – Loss of Anchorage of Stirrups formed by Lapped U-bars

(Adapted from Law C.W., Mak J.Y.W., "Some explanatory/commentary notes on reinforced concrete detailing in accordance with the Code of Practice for Structural Use of Concrete 2004", The HKIE Transactions Vol. 20 No. 1, 2013, pp34-51)

## 9.3 Solid slabs

### 9.3.1 Flexural reinforcement

The minimum tension reinforcement to be provided in each direction shall be the same as that for rectangular beams.

In one-way slabs, secondary transverse reinforcement of not less than 20% of the principal reinforcement should be provided. In areas near supports, transverse reinforcement to the principal top bars needs not be provided where there is no transverse bending moment.

The maximum spacing of reinforcing bars in slabs should comply with the following requirements, which are adopted from Eurocode 2 ($h$ is the total depth of slab):

In general areas without concentrated loads:

- for the principal reinforcement, max. spacing ≤ 3$h$ ≤ 400 mm; and
- for the secondary reinforcement, max. spacing ≤ 3.5$h$ ≤ 450 mm.

In areas with concentrated loads or areas of maximum moment:

- for the principal reinforcement, max. spacing ≤ 2$h$ ≤ 250 mm; and
- for the secondary reinforcement, max. spacing ≤ 3$h$ ≤ 400 mm.

It is also suggested in the code that no further check is required on bar spacing if either (this is derived from BS8110: Part 1: 1997):

- max. spacing ≤ $h$ ≤ 250 mm (grade 250 steel);

- max. spacing ≤ $h$ ≤ 200 mm (grade 500 steel); or
- the percentage of required tension reinforcement is less than 0.3%.

Where none of these conditions applies, the bar spacings should be limited to the values calculated in Clause 9.2.1.4 of the code.
Curtailment and anchorage of tension reinforcement in slabs may be carried out in accordance with Clause 9.2.1.6 of the code as for tension reinforcement in beams.

At an end support, which acts as a simple support, at least 50% of the calculated mid-span bottom reinforcement should be extended to the end support and anchored therein in accordance with Section 8.4 and Clause 9.2.1.7 as for beams. If the design ultimate shear stress at the face of the support is less than half of $v_c$, a straight length of bar beyond the centreline of the support equal to 1/3 of the width of support or 30 mm, whichever is the greater, may be considered as effective anchorage.

Hogging moments due to partial fixity of a support could lead to cracking. To control this, an amount of top reinforcement capable of resisting 50% of the maximum mid-span sagging moment should be provided at the support and the top reinforcement so provided should have an effective anchorage into the support and extend not less than 0.15 times the span length or 45 times the bar diameter into the span. As before, the author does not recommend such practice of neglecting the partial fixity in the design calculations because this could lead to cracking of the joint between the support and the slab. The partial fixity should be taken into account in the design calculations and the hogging moment so induced properly designed for.

At an intermediate support of a continuous slab, at least 40% of the calculated mid-span bottom reinforcement should be continuous through the support.

If the detailing arrangements are such that lifting of a corner is restrained, suitable reinforcement should be provided, as required by Clause 6.1.3.3 of the code.

Along a free edge, the slab should be provided with both longitudinal and transverse reinforcement, as shown in Figure 9.4 of the code.

### 9.3.2 Shear reinforcement

Shear reinforcement, if required, should be provided in accordance with Section 6.1 of the code. However, it should be noted that it is in general difficult to bend and fix shear reinforcement and assure its effectiveness in a thin slab, especially if the slab is less than 200 mm thick.

## 9.4 Cantilevered projecting structures

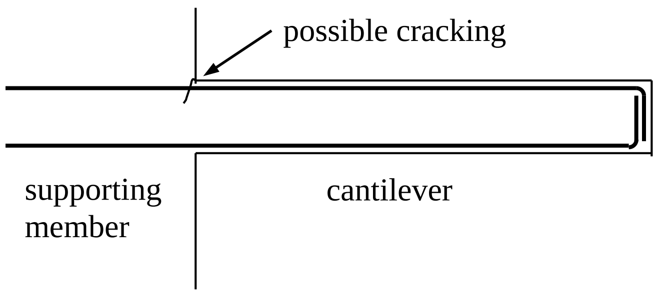
The code specifies a minimum top tension reinforcement of 0.25%, a minimum bar size of 10 mm and a maximum bar spacing of 150 mm. All the top tension reinforcement should be anchored with a full anchorage length beyond the centre line of the supporting member and no reduction in anchorage length due to actual bar stress should be permitted. Apart from complying with these requirements, the author suggests that particular attention should be paid to the risk of cracking at the top surface of the root of the cantilever due to stress concentration. Stress concentration and risk of cracking at the root of the cantilever can be reduced by (see Figure 9.5):

- adding a splay underneath the root of the cantilever; or
- adding a splay with diagonal tie back bars inside on top of the root of the cantilever;

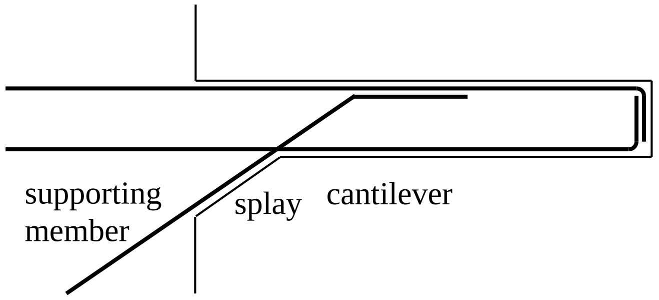
so that the pull out stresses of the top tension reinforcement and the tensile stresses of concrete are reduced. Good drainage should also be provided to avoid trapping of moisture at the top surface of the root of the cantilever, which may cause corrosion of the top tension reinforcement there.

Instead of pure cantilevered slab arrangement, beam-and-slab arrangement should be used for spans exceeding 1200 mm. A cantilevered structure should have the following requirements and the requirements of clause 7.3 are complied with:

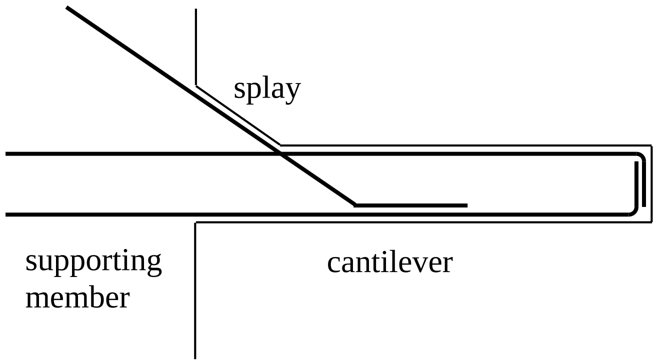
(a) 300 mm at the support of cantilevered beam;
(b) 110 mm for cantilevered slab with span not exceeding 500 mm;
(c) 125 mm for cantilevered slab with span greater than 500 mm but not exceeding 750 mm;
(d) 150 mm for cantilevered slab with span greater than 750 mm but not exceeding 1000 mm;
(e) 175 mm for cantilevered slab with span exceeding 1000 mm but not exceeding 1200 mm.




(a) Possible cracking at root of cantilever




(b) Adding a splay underneath the root




(c) Adding a splay on top of root

Figure 9.5: Adding splays to avoid cracking of cantilever structures

## 9.5 Columns

### 9.5.1 Longitudinal reinforcement

The minimum requirements for longitudinal reinforcement are:

- minimum total area: 0.8%;
- minimum bar size: 12 mm;
- minimum number of bars: 4 in rectangular columns or 6 in circular columns;
- at least one bar at each corner of a polygonal shaped column.

To avoid steel congestion, the total area of longitudinal reinforcement should not exceed:

- vertically cast columns: 6%;
- horizontally cast columns: 8%; and
- laps in vertically or horizontally cast columns: 10%.

To avoid spalling of concrete cover, the sum of the bar sizes of the longitudinal reinforcement in a particular layer along the perimeter of the column should not exceed 40% of the perimeter of the column, even at lap locations.

### 9.5.2 Transverse reinforcement

The transverse reinforcement is to provide nominal confinement to the concrete in the core and to prevent the longitudinal bars from buckling. It may be in the form of links, loops or helical spiral reinforcement.

The minimum requirements for transverse reinforcement are:

- minimum bar size: 1/4 of the largest longitudinal bar but not less than 6 mm;
- maximum spacing: The least of 12 times the diameter of the smallest longitudinal bar, the lesser dimension of the column and 400 mm. The requirements tally with those stated in Eurocode 2.

*Rectangular or polygonal columns:*
Links should be provided to restrain every longitudinal bar at a corner and each alternate longitudinal bar in an outer layer of reinforcement. If necessary, additional links should be provided such that no longitudinal bar within the compression zone is further than 150 mm from a restrained bar. To provide restraint, the link should pass around the longitudinal bar with an included angle of not more than 135$^o$. The links should be anchored by means of hooks bent through an angle of at least 135$^o$ into the concrete core. Crossties should be adequately anchored by means of hooks bent through an angle of not less than 135$^o$ at one end and 90$^o$ at the other end and should be alternated end for end along the longitudinal bars. It has been shown that the ductility and energy dissipation capacity of column is enhanced with restrained longitudinal reinforcement. Readers may refer to a paper for details: Kuang, J.S., Wong, H.F., "Improving ductility of non-seismically designed RC columns", Proceedings of Civil Engineers – Structures and Building*s*, Vol. 158, issue 1, 2005, pp13-20.

Where there is adequate confinement to prevent the end anchorage of the link from "kick off" (Figure 9.6), the 135$^{o}$ hook in the links or crossties may be replaced by other standard hoods given in Figure 8.2 of the Code.

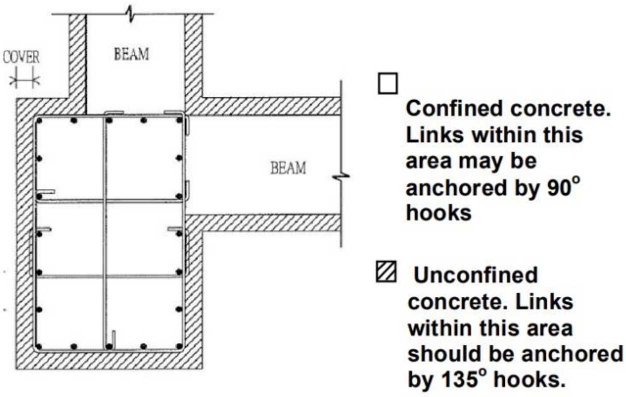


Figure 9.6: Column transverse reinforcement

*Circular columns:*
Loops or spiral reinforcement satisfying the above minimum requirements should be provided. Loops (circular links) should be anchored with a mechanical connection or a welded lap or by terminating each end with a 135$^{o}$ hook bent around a longitudinal bar after overlapping the other end of the loop by a minimum length. Spirals should be anchored either by welding to the previous turn or by terminating each end with a 135$^{o}$ hook bent around a longitudinal bar and at not more than 25 mm from the previous turn. Loops and spirals should not be anchored by straight lapping, which causes spalling of the concrete cover.

## 9.6 Walls

### 9.6.1 General

For walls subjected only to in-plane forces, the provisions in this section apply. For walls subjected to both in-plane and out-of-plane forces, both the provisions in this section and the provisions for slabs in Section 9.3 apply.

### 9.6.2 Vertical reinforcement

The minimum total area of vertical reinforcement is 0.4%. Where the minimum area of reinforcement controls the design, half of the area should be located at each face so that there would be at least 0.2% vertical reinforcement at each face to resist expected or unexpected out-of-plane bending and to control cracking. To control cracking, the maximum spacing between vertical bars should be 3 times the wall thickness but not larger than 400 mm. To avoid steel congestion, the total area of vertical reinforcement should not exceed 4%. This limit may be doubled at laps, which tallies with the Clause 9.6.2(1) of Eurocode 2.

9.6.3 Horizontal reinforcement

Where the vertical compression reinforcement $\leq 0.2\%$, horizontal reinforcement should be provided such that:

- total area: $\geq 0.30\%$ for $f_y = 250\,\mathrm{N/mm^2}$ or $\geq 0.25\%$ for $f_y = 500\,\mathrm{N/mm^2}$;
- minimum bar size: 1/4 of the largest vertical bar size but not less than 6 mm;
- maximum spacing: 400 mm.

9.6.4 Transverse reinforcement

Where the vertical compression reinforcement > 2%, transverse reinforcement in the form of links should be provided such that:

- minimum bar size: 1/4 of the largest vertical bar size but not less than 6 mm;
- maximum vertical spacing: 2 times the wall thickness or 16 times the largest vertical bar size, whichever is the smaller;
- maximum horizontal spacing: 2 times the wall thickness;
- every vertical compression bar is enclosed by a link;
- no vertical compression bar is further than 200 mm from a restrained vertical bar;
- at a restrained vertical bar, the link should pass round the bar with an included angle of not more than $90^{\circ}$.

9.6.5 Plain walls

To provide nominal resistance and to control cracking, minimum reinforcement in both the vertical and horizontal directions should be provided.

## 9.7 Foundations

9.7.1 Pile caps

The main tension reinforcement should be concentrated in the stress zones between the tops of the piles. Check should be made to ensure that the total area of tension reinforcement is not less than the minimum required as stipulated in Table 9.1 of the code.

The code suggests that the sides and top surfaces may be unreinforced if there is no risk of tension developing in these areas. However, it is difficult to judge whether there is any risk of tension developing in any parts of a pile cap. A pile cap is normally quite thick and rigid. Any small vertical movement of the piles could induce a relatively large sagging or hogging moment onto the pile cap structure. Because of such high sensitivity to small vertical movement of the piles, we are actually never sure of the risk of tension developing in any particular part of the pile cap. Moreover, a pile cap is usually a massive concrete structure and there is always the risk of early thermal cracking. Hence, the author would not recommend having any surface of a pile cap totally unreinforced. At any surface, at least the minimum reinforcement stipulated in Table 9.1 should be provided.

A pile cap is actually a transfer structure. Its design is usually governed by shear. However, the provision of shear reinforcement in a pile cap is generally both difficult and uneconomical. It should be better and more economical to provide a sufficient thickness for the pile cap so that no shear reinforcement would be necessary.

### 9.7.2 Column and wall footings

Tension reinforcement should be provided where necessary. Check should be made to ensure that the total area of tension reinforcement is not less than the minimum required as stipulated in Table 9.1 of the code.

As before, the author would not recommend having any surface of a footing totally unreinforced. At any surface, at least the minimum reinforcement stipulated in Table 9.1 should be provided.

### 9.7.3 Tie beams

Tie beams are not just to tie the individual footings or caps together. Their purpose is to integrate the footings / caps to bend as a group so as to reduce the eccentricity of the loadings. In doing so, they would be subjected to bending moments and shear forces. Hence, they should be designed as beams rather than as ties. If the action of compaction machinery could affect the tie beams, the tie beams should be designed for a minimum downward load of 10 kN/m.

### 9.7.4 Bored piles and baraettes

Reinforcement would be provided to bored piles and baraettes in order to resist any combination of applied load e.g. dead load, imposed load and wind load. The code provides minimum steel ratio which is dependent on the pile cross-sectional area. The reference is made from BS EN1536:2000 - Execution of special geotechnical work. Bored piles.

## 9.8 Corbels

The design of corbels may be based on a strut-and-tie model satisfying the equilibrium and strength requirements. At the front face of the corbel, the reinforcement should be anchored either by welding to a transverse bar or bending the bars back to form a loop. Shear reinforcement should be provided in the form of horizontal links, as shown in Figure 9.6 of the code. If there is any horizontal force, additional reinforcement connected to the supporting member should be provided to transmit this force in its entirety.

## 9.9 Detailing for ductility

The ductility of a reinforced concrete member is dependent on many factors, including the following:

- The strength grade of the concrete: As a material, a higher strength concrete is generally more brittle, but the increase in concrete strength could increase the capacity of the concrete member and reduce the load to capacity ratio. If the size of the member is not reduced to take advantage of the increased concrete strength, the use of a higher strength concrete could slightly increase the ductility of member. But if the size of the member is reduced to take advantage of the increased concrete strength, then the increase or decrease in ductility of the member is dependent on how much the member size has been reduced; in such case, detailed analysis needs to be carried out to evaluate the actual change in ductility of the member.
- The amounts of tension and compression reinforcement provided: For a flexural member, the degree of reinforcement (or more precisely, the $(\rho_t - \rho_c)/\rho_b$ ratio in which $\rho_t$, $\rho_c$ and $\rho_b$ are the tension, compression and balanced steel ratios, respectively) is the major factor governing the flexural ductility of the member. A higher degree of reinforcement would lead to a lower ductility and a lower degree of reinforcement would lead to a higher ductility. That is why an under-reinforced member is more ductile than an over-reinforced member and over-reinforcement should be avoided. As a general rule, since a lower tension steel ratio would give a higher ductility, the tension steel ratio should be limited. On the other hand, since a higher compression steel ratio would give a higher ductility, the addition of more compression steel could help to improve the flexural ductility of the member.
- The amount of confining reinforcement provided: With at least some confining reinforcement provided, the concrete would be subjected to triaxial compression instead of uniaxial compression. Under triaxial compression, both the compressive strength and ductility of the concrete would be significantly increased. Hence, it should be a good practice to always provide some nominal confining reinforcement to the compression zones of the member. This is particularly so when the member is cast of high-strength concrete. For all high-strength concrete members, therefore, it should be

advisable to always provide generous confining reinforcement to the compression zone in the case of a beam and to the whole member in the case of a column.

- Anchorage of the reinforcing bars: Anchorage failure of reinforcing bars would lead to sudden and brittle failure. Most codes just demand anchorage lengths corresponding to the design strengths of the steel bars (i.e. factored down yield strengths of $0.87f_y$ or $0.95f_y$) to be provided. However, this could cause the bars to fail by anchorage slip before yielding. It should have been much better if the reinforcing bars are always designed to yield before any possible anchorage failure so as to fully utilize the yield strength and ductility of each steel bar. Provision of better anchorages to reinforcing bars to the extent that even when the bars yield and reach the strain hardening stage the bars would not fail by anchorage slip would help to improve the ductility of the member.
- Stability of compression bars: Buckling failure of compression bars would also lead to sudden and brittle failure. Hence, all compression bars should be properly contained and restrained by transverse reinforcement to prevent buckling.
- Containment of bursting forces: Bursting forces inside the concrete could arise from: (1) the concentrated forces at the anchorages of prestressing tendons, (2) the bond forces of relatively large diameter bars, (3) the end bearing stresses of compression bars, and (4) the bearing stresses from other parts of the structure. All these bursting forces should be properly contained by transverse reinforcement or otherwise there would be splitting or spalling failure of the surrounding concrete.
- Restraint of radial forces at where tendons or reinforcing bars change directions: At where tendons or reinforcing bars change directions (i.e. at curves and bends), radial forces would be developed and if these radial forces are acting outwards and only resisted by thin concrete layers or covers, the concrete would fail by spalling. To prevent such possible spalling failure of concrete, restraining reinforcement should be provided to tie all the curving tendons and reinforcing bars back into the core of the member. If practicable, all curves and bends of tendons/reinforcing bars should be curving inwards into the core of the member so that the radial forces would only act inwards.

Ductility is needed not only for earthquake resistance, but also for general structural safety with respect to impact loads, cyclic loads and accidental loads. In actual fact, ductility helps to redistribute the loads from an overloaded and yielded member to the other parts of the structure so that even when a member has been overloaded, it would not collapse immediately. This applies to all kinds of loading and hence ductility helps to promote structural safety regardless of the type of loading applied; in other words, ductility is crucial to the safety of a structure even when the structure is just subjected to ordinary dead, imposed and wind loads.

From the structural safety point of view, ductility is at least as important as strength, but most engineers tend to pay more attention to strength than to ductility. The inclusion of this section in the new code to encourage more refined reinforcement detailing for better ductility of reinforced concrete members and eventually higher survivability and safety of the structure as a whole should be highly commended.

### 9.9.1 Beams

*Longitudinal reinforcement:*

For improved ductility, the minimum and maximum tension reinforcement within the critical zone should be changed to 0.3% and 2.5%, respectively. At any section within a critical zone, e.g. a potential plastic hinge zone, the compression reinforcement should not be less than 1/2 of the tension reinforcement at the same section.

The limitation of tension reinforcement ratio to 2.5% is identical to that of the New Zealand Concrete Code NZS3101-2006 for provision of ductility in beam, as per the phenomenon that increase of tension reinforcement will reduce beam ductility.

The extent of critical zone should be from the column face over a length equal to 2 times the beam depth. For other special beam configurations such as haunched beam, the critical zone may be demonstrated to suit the possible location of plastic hinge.

Where longitudinal bars in a beam are anchored into the core of an exterior column, the anchorage should commence at 1/2 of the depth of the column or 8 times the bar diameter from the face at which the bars enter the column. For the calculation of anchorage length, the bars should be assumed to be fully stressed to at least their actual yield strengths. This would ensure that the bars would yield before any anchorage failure. No beam bar should be terminated in a joint area without a vertical $90^{o}$ standard hook or an equivalent anchorage device as near as practically possible to the far side of the column core and not closer than 3/4 of the depth of the column to the face of entry. Top beam bars should only be bent down and bottom beam bars should only be bent up. Bending top beam bars up and bottom beam bars down would greatly reduce the joint shear strength and joint shear failure may occur before reaching design flexural capacity of beams. Bending both top and bottom beam bars up or down also results in poor structural performance for joint. Readers may refer to a paper for details: Kuang, J.S., Wong, H.F., "Effects of beam bar anchorage on beam-column joint behaviour", Proceedings of Institute of Civil Engineers – Structures and Buildings, Vol. 159, Issue 2, 2006 pp.115-124.

Full strength welded splices may be used in any location, but laps and mechanical couplers should not be located within the beam-column joint region or within one effective depth of the member from the critical section of a potential plastic hinge where stress reversals could occur. No beam bar should be lapped in a region where reversing stress at the ultimate limit state may exceed $0.6f_y$ unless each lapped bar is confined by links or ties satisfying Equation 9.6.

*Transverse reinforcement:*

Links or ties should be arranged so that every corner and alternate longitudinal bar that is required to function as compression reinforcement would be restrained by a leg. Refer to NZS, the centre-to-centre spacing of links or ties along a beam shall not exceed:

(a) inside the critical zone: the larger of 150mm or 8 times the longitudinal bar diameter
(b) outside the critical zone: the smaller of the least lateral dimension of the cross section of the beam or 12 times the longitudinal bar diameter.

Clause 9.9.1.3 stipulate that the spacing of transverse reinforcement inside the critical zone should not exceed the larger of 150 mm or 8 times the diameter of longitudinal bar; all links should be anchored by means of 135$^o$ or 180$^o$ hooks. Anchorage by means of welded cross bars should not be permitted. Where there is adequate confinement to prevent the end anchorage of the link from "kick off" (Figure 9.7), the 135$^{\circ}$ hook may be replaced by other standard hoods given in Figure 8.2 in the code of practice.

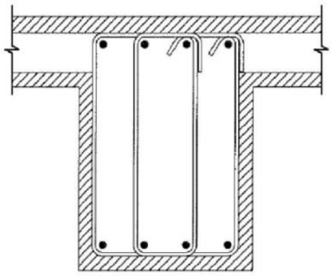



Figure 9.7: Typical confinement in beam

9.9.2 Columns

*Longitudinal reinforcement:*
The minimum and maximum longitudinal reinforcement should be changed to 0.8% and 4%, respectively. In any row of longitudinal bars, the smallest bar diameter should not be less than 2/3 of the largest bar diameter. Where the column bars pass through the beams at beam-column joints, the bar diameter should be limited as per Equation 9.7. In potential plastic hinge regions, the cross-linked bars, i.e. the transverse reinforcement restraining the longitudinal bars should not be spaced further apart between centres than the larger of 1/4 of the adjacent column dimension or 200 mm. Where column bars terminate in beam-column joints or joints between columns and foundation members and where a plastic hinge in the column could be formed, the anchorage of the column bars into the joint region should commence at 1/2 of the depth of the beam/foundation member or 8 times the bar diameter from the face at which the bars enter the beam or the foundation member. No column bar should be terminated in a joint area without a horizontal 90$^o$ standard hook or an equivalent anchorage device as near as practically possible to the far side of the beam and not closer than 3/4 of the depth of the beam to the face of entry. Unless the column is designed to resist only axial force, the direction of the horizontal leg of the bend should be towards the far face of the column. Full strength welded splices may be used in any location, but centre of the laps length and mechanical type 1 couplers should be located at a height not below (storey height)/4 above floor level unless it can be

shown that no plastic hinge can be developed in the column or the condition $\sum M_c \geq 1.2 \sum M_b$ is complied with. Type 1 mechanical couplers should also be staggered in at least 2 layers at not less than 900mm apart.

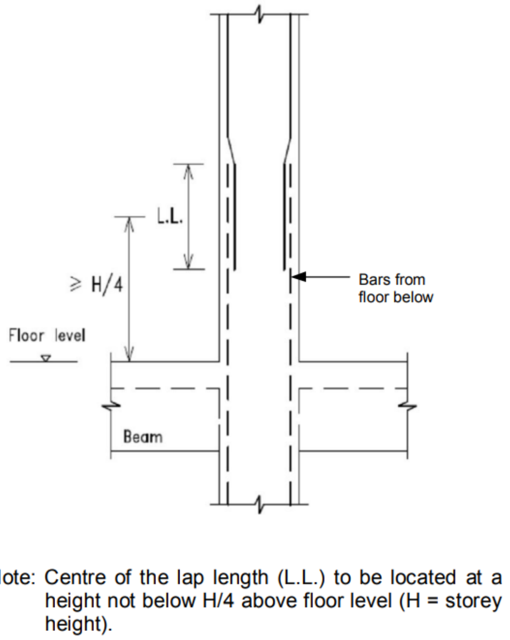



Figure 9.8: Bar lapping details for column

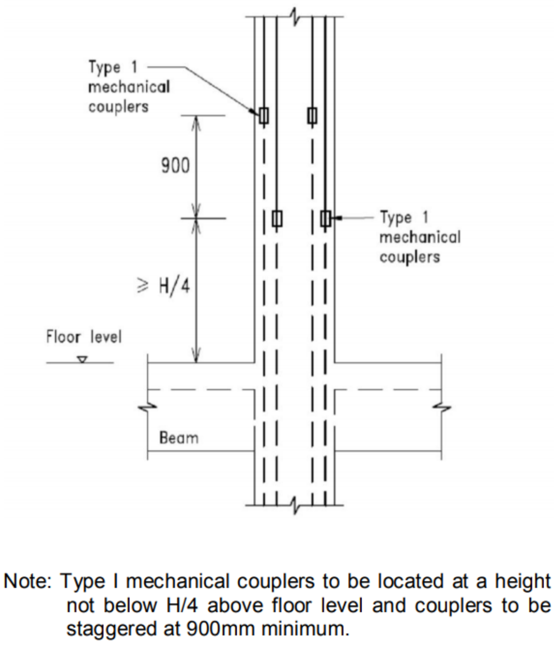



Figure 9.9 Type 1 mechanical coupler details for column

Type 2 mechanical couplers should be staggered in at least 2 layers at not less than 300 mm apart. The lowest layer should not be less than 300 mm above the top level of structural floor, pile cap or transfer structure.

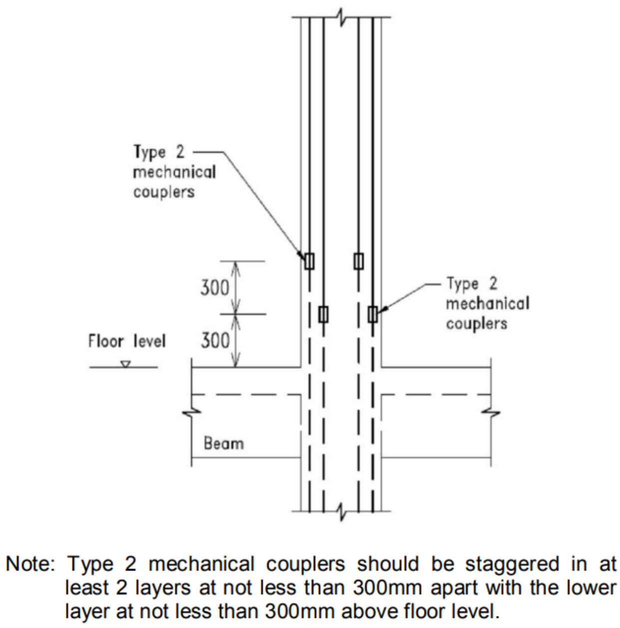



Figure 9.10: Type 2 mechanical coupler details for column

*Transverse reinforcement within critical regions:*

The critical zone in a reinforced concrete column is considered as a region where closely-spaced transverse reinforcement in the form of hoops, cross-ties or spirals is required for higher ductility to safeguard the column. In general, the length of confinement zone, which is also known as the extent of critical zone, is related directly to the length of the plastic hinge, which can be defined as the physical region where the column experiences inelastic deformations and severe damage. The extent of critical region in limited ductility high-strength reinforced concrete columns should be from the point of maximum moment over a finite length as depicted below ($N$ is the axial load and $A_g$ is the gross area of section):

- For $0 < N/A_g f_{cu} \leq 0.1$, the extent of critical region is taken as 1.0 times the greater dimension of the section or where the moment exceeds 0.85 of the maximum moment or 1/6 of column clear height at floor, whichever is larger.
- For $0.1 < N/A_g f_{cu} \leq 0.3$, the extent of critical region is taken as 1.5 times the greater dimension of the section or where the moment exceeds 0.75 of the maximum moment or 1/6 of column clear height at floor, whichever is larger.
- For $0.3 < N/A_g f_{cu} \leq 0.6$, the extent of critical region is taken as 2.0 times the greater dimension of the section or where the moment exceeds 0.65 of the maximum moment or 1/6 of column clear height at floor, whichever is larger.

Comparing with NZS 3101:2006, other than the newly introduced value $1/6\, l_0$, the code generally releases the requirements for the extent of the critical or potential plastic hinge regions, which are to be detailed for confining the concrete and restraining the longitudinal reinforcement buckling. The ceiling of the critical zone extent stipulated in the code is only two times the greater dimension of the column cross-section, while in the NZS 3101:2006, it can be as large as three times. In case axial load effect is in between 60% - 70%, the reference could be referred to NZS 3101:2006.

For the other codes provisions on the extents of critical regions in RC column, readers could refer to the paper, Yuen, T.Y.P., Kuang, J.S. and Ho, D.Y.B., “Ductility design of RC columns. Part 2: extent of critical zone and confinement reinforcement”, HKIE transactions, Vol. 24, No. 1, 2017, pp 42-53.

The transverse reinforcement should be arranged so that each longitudinal bar or bundle of bars would be restrained by a leg of the transverse reinforcement. The diameter of the transverse reinforcement (links, loops or spirals) should not be less than 10 mm or 1/4 of the diameter of the largest longitudinal bar, whichever is the greater. For rectangular or polygonal columns the centre-to-centre spacing of links or cross-ties along a column shall not exceed the smaller of 8 times the diameter of the longitudinal bar to be restrained or 150mm. The authors recommend that the largest longitudinal bar diameter to be considered. The arrangement of links or ties within the cross section shall comply with either one of the following requirements:

(i) each longitudinal bar or bundle of bars shall be laterally supported by a link passing around the bar, or

(ii) every corner bar and each alternate longitudinal bar (or bundle) in the outer layer of reinforcement shall be supported by a link passing around the bar, and no bar within the compression zone shall be further than the smaller of 10 times the diameter of link or 125mm from a restrained bar.

For the above arrangement, reference has been made to Clause 11.4.12 of GB50010-2002.

*Transverse reinforcement outside critical regions:*
Outside the critical regions, the normal requirements in accordance with Clause 9.5.2 of the code would apply.

### 9.9.3 Wall

In addition to columns, walls also needs to consider the effect of critical zone and confined boundary is incorporated. Depends on the height of wall, the critical zone are defined as below:

(a) Walls with height not exceeding 24 m

The critical zone is from support to ceiling of the lowest floor.

(b) Walls with height exceeding 24 m

The critical zone extends from support to the ceiling of second lowest floor or 1/10 of full height of wall, whichever is the greatest.

Three different types of confined boundary elements with different reinforcement ratio, minimum bar diameter and spacing are defined.

The critical zone and remaining storeys are then incorporated with various types of confined boundary elements dependent on the value of axial compression ratio $N_{cr}$ below:

<u>For walls with $0 < N_{cr} \leq 0.38$:</u>
Critical zone is strengthened with type 2 confined boundary element. The remaining storeys are strengthened by type 1 element.

<u>For walls with $0.38 < N_{cr} \leq 0.75$:</u>
Critical zone and the storey immediately above is strengthened with type 3 confined boundary element. All other storeys are strengthened by type 1 element.

In no case should $N_{cr}$ exceed 0.75.

The code requirements are developed based on GB50011-2010 Clause 6.4.1 to 6.4.5 with modification.

# 10 GENERAL SPECIFICATION, CONSTRUCTION AND WORKMANSHIP

## 10.1 Objectives

The design assumptions of the code are based on the required standards for materials and workmanship specified in this chapter. These required standards should be clearly and unambiguously stipulated in the specifications or noted on the drawings and appropriate site supervision and compliance testing should be arranged to ensure their strict adherence.

## 10.2 Construction tolerances

A long list of permissible deviations based on the accuracy that can normally be achieved has been given. More stringent dimensional accuracy requirements may be necessary if the performance of the structure to be constructed is particularly sensitive to small deviations in dimension, position, verticality or fitting between elements (especially between prefabricated elements).

The dimensional accuracy of in-situ concrete construction is dependent mainly on the type of formwork (e.g. whether of steel or timber construction), the accuracy and rigidity of falsework, workmanship, site supervision and sometimes the working conditions. For superstructures, the deviation is normally less than 25 mm while for substructures, the deviation tends to be larger and can be up to 50 mm. For piles, the deviation can amount to more than 100 mm.

## 10.3 Concrete

### 10.3.1 Constituents

The constituent materials should comply with the acceptable standards, as listed in Annex A of the code.

### 10.3.2 Mix specification

The recommended methods given in the list of acceptable standards in Annex A of the code should be followed. Each mix should be identified by a distinct mix number. For an approved mix identified by a registered mix number, only minor variations in the mix proportions, such as slight changes in the dosages of retarder and superplasticizer to cater for differences in ambient temperature and transportation time, should be allowed. If a relatively large variation in the mix proportions is unavoidable, the varied mix should be assigned a different mix number and separate approval should be sought before use.

10.3.3 Methods of specification, production control and transport

The recommended methods given in the list of acceptable standards in Annex A of the code should be followed.

10.3.4 Sampling, testing and assessing conformity

The specifications given in Hong Kong Construction Standard CS1 and the list of acceptable standards in Annex A should be adopted.

*Additional cubes:*
For special purposes, such as checking the strength of concrete in prestressed concrete at transfer and determining the time to strike formwork, additional cubes may be cast for testing. These additional cubes should be made and tested in accordance with CS1 but the methods of sampling and the conditions of storage and curing should be specified to meet with the required purpose. Preferably, sampling should be at the point of placing and the additional cubes stored and cured under the same conditions as the concrete in the members. These additional cubes should not be used for compliance testing.

*Concrete cubes for compliance testing:*
The compressive strength of concrete shall be determined by testing 100 mm or 150 mm cubes at the age of 28-day after mixing. A representative sample shall be taken from the fresh concrete to make the cubes. Each sample shall be taken from a single batch. The rate of sampling shall be at least that specified in Table 10.1 of the code and at least one sample shall be taken from each mix (as identified by a distinct mix number) produced on any one day. From each sample of fresh concrete taken, 2 cubes shall be made. Each cube shall be given a cube number sequentially and no cube number shall be duplicated or omitted. All cubes shall be adequately cured until ready for testing at the age of 28-day. The average compressive strength of each pair of cubes made from the sample shall be taken as the test result.

*Acceptance criteria for 150 mm cubes:*
Where the standard deviation of 40 previous consecutive test results is not less than 5 MPa or is not known, the compliance requirement C1 shall be adopted. Where there is sufficient previous production data using similar materials from the same plant under similar supervision to establish that the standard deviation of 40 test results is less than 5 MPa or where the standard deviation of 40 previous consecutive test results is less than 5 MPa, the compliance requirement C2 shall be adopted. For concrete of grade 20D or above, C1 requires that each individual test result shall not be less than the specified grade strength minus 3 MPa and the average of 4 consecutive test results shall not be less than the specified grade strength plus 5 MPa whereas C2 requires that each individual test result shall not be less than the specified grade strength minus 3 MPa and the average of 4 consecutive test results shall not be less than the specified grade strength plus 3 MPa. For concrete of grade lower than 20D, each individual test result shall not

be less than the specified grade strength minus 2 MPa and the average of 4 consecutive test results shall not be less than the specified grade strength plus 2 MPa. When there are less than 4 test results, the test results shall be treated as if they were 4 consecutive test results. When there are 4 or more test results, the average of each set of 4 consecutive test results shall be calculated to check for compliance each time a new test result is produced using that test result and the immediately preceding 3 test results. If the above requirements are not satisfied by any test result, investigation shall be carried out to establish whether the concrete represented by the failed test result is acceptable or not.

*Acceptance criteria for 100 mm cubes:*
Where the standard deviation of 40 previous consecutive test results is not less than 5.5 MPa or is not known, the compliance requirement C1 shall be adopted. Where there is sufficient previous production data using similar materials from the same plant under similar supervision to establish that the standard deviation of 40 test results is less than 5.5 MPa or where the standard deviation of 40 previous consecutive test results is less than 5.5 MPa, the compliance requirement C2 shall be adopted. For concrete of grade 20D or above, C1 requires that each individual test result shall not be less than the specified grade strength minus 2 MPa and the average of 4 consecutive test results shall not be less than the specified grade strength plus 7 MPa whereas C2 requires that each individual test result shall not be less than the specified grade strength minus 2 MPa and the average of 4 consecutive test results shall not be less than the specified grade strength plus 5 MPa. For concrete of grade lower than 20D, each individual test result shall not be less than the specified grade strength minus 2 MPa and the average of 4 consecutive test results shall not be less than the specified grade strength plus 3 MPa. When there are less than 4 test results, the test results shall be treated as if they were 4 consecutive test results. When there are 4 or more test results, the average of each set of 4 consecutive test results shall be calculated to check for compliance each time a new test result is produced using that test result and the immediately preceding 3 test results. If the above requirements are not satisfied by any test result, investigation shall be carried out to establish whether the concrete represented by the failed test result is acceptable or not.

*150 mm cubes versus 100 mm cubes:*
Most of the testing laboratories in Hong Kong are equipped with compression testing machines with a maximum loading capacity of around 2000 kN. These machines are capable of testing 150 mm cubes up to a concrete strength of 90 MPa. With the development of high-strength concrete, the capability of these testing machines may not be sufficient. The wear and tear of the machine platens under high loading will also be very severe. The use of 100 mm cubes for compliance testing could alleviate such problems. The 100 mm cubes would be easier to handle and could result in saving of materials and labour. The storage and curing space, as well as the testing time, can also be reduced. As a matter of fact, the Mass Transit Railway Corporation and the local concrete producers have specified the use of 100 mm cubes for internal quality control. In 1993, the Standing Committee on Concrete Technology (SCCT) requested the Public Works Central Laboratories (PWCL) to carry out an investigation to examine the suitability of using 100 mm cubes for concrete quality control purposes. The general conclusions are: (1) the 28-day strength of the 100 mm cubes is on average higher

than that of the 150 mm cubes made from the same batch of concrete; and (2) the standard deviation of the 100 mm cubes is on average higher than that of the 150 mm cubes. Since then, the Architectural Services Department has used 100 mm cubes for its projects for concrete with maximum aggregate size not exceeding 20 mm. In November 2000, the then Civil Engineering Department promulgated the use of 100 mm cubes in Technical Circular No.5/2000. The Housing Department also proposed the use of 100 mm cubes for projects tendered out after October 2001 based on their study on the use of 100 mm cubes in housing projects conducted from 1998 to 1999.

10.3.5 Placing and compacting

To ensure good compaction so as to produce a dense and uniform concrete with no blowholes or honeycombs formed, the fresh concrete should possess suitable workability and appropriate placing procedures and compacting equipment should be used.

The workability of a concrete mix is a broad term for describing the relative ease of working with the fresh concrete mix to achieve full and uniform consolidation inside the mould. In practice, working with a fresh concrete mix includes placing, spreading around, passing through gaps, ensuring uniform distribution of the ingredients, driving away entrapped air and levelling of the concrete mix. Hence, there cannot be a single measure of workability. The workability of a concrete mix should include at least the following attributes:

- consistence;
- deformability/flowability;
- passing ability; and
- segregation resistance (cohesiveness).

The *consistence* of the concrete mix is dependent mainly on the degree of dryness/wetness of the concrete mix. When dry powders are added and mixed with a small amount of water, loose lumps of slightly wetted powders would be formed. These lumps have rather low cohesiveness and tend to break into smaller lumps when disturbed. By adding more water, the lumps of powders formed would grow in size and become more cohesive. As more and more water is added, there is a certain stage that all the lumps formed could be coalesced into a paste. At such stage, the paste is nearly saturated inside but because of the surface tension forces that pull the water inwards, the surfaces of the paste remain dry. Such a paste formed is quite cohesive but is rather stiff (i.e. it has no deformability and no flowability). Some researchers call the water content at this stage the water demand of the given mixture of powders and use this water content to determine the amount of voids in the bulk volume of the given mixture of powders (it is generally believed that at this stage the amount of water added is just enough to fill up the voids in the bulk volume of powders).

If, after forming a paste, additional water is added, then gradually the paste would become less stiff and both its *deformability* and *flowability* would increase. It is the deformability and flowability that imparts the mixture of powders the conventional sense of workability. Both the deformability and flowability would increase with the water content. But when the water content is increased to a certain level, the mixture would become completely deformable and further increase in the water content would not produce any significant increase in deformability. It is noteworthy that the slump (reduction in height) of a concrete mix measured during the conventional slump test is actually only a measure of the deformability of the concrete mix. When water has been added to the level that the slump of the concrete mix is already 220 or 230 mm, further addition of water would at the most increase the slump marginally by 20 to 30 mm. However, even at this stage, the flowability of the mixture would continue to increase with the water content. In the context of concrete technology, the flowability of a concrete mix may be measured in terms of the average diameter of the slumped concrete mix (also called flow diameter) as part of the slump test. This flow measurement method is gaining popularity in the field of self-consolidating concrete. Summing up, it may be concluded that the deformability is an appropriate measure of workability for low to medium workability concrete while the flowability is a more appropriate measure of workability for high workability concrete. In practical terms, if the slump test is used to measure workability, the slump is an acceptable measure of workability when the slump is less than 200 mm but the flow diameter is a better measure of workability when the slump is larger than 200 mm.

The *passing ability*, i.e. the ability of the concrete mix to pass through narrow gaps, is also an important attribute of the workability of the concrete mix. The concept of passing ability emerges from recent research on self-consolidating concrete but the concept is applicable also to ordinary concrete. Whilst the passing ability can be improved by increasing the deformability and flowability of the concrete mix, it is affected and in fact severely limited by the width of gap that the concrete mix has to pass through. The passing ability of a concrete mix is very much dependent on the gap width to maximum size of aggregate ratio (the gap width is either the clear distance between reinforcing bars or the thickness of concrete cover). Where this ratio is smaller than 1.5, there is likely to be passing ability problem and particular attention is needed during compaction to ensure that no voids are formed inside the mould due to jamming of the concrete mix at narrow gaps. Where this ratio is smaller than 1.3, the risk of having passing ability problem is even higher. In such case, extreme care is needed during compaction to ensure that no voids are formed inside the mould and it may be advisable to limit the content of the largest size aggregate to say not more than 25% by volume. For instance, if the smallest gap width is 25 mm and the maximum size of aggregate is 20 mm, then apart from taking the necessary care during compaction, the content of the 10 - 20 mm aggregate should be limited to a maximum of 25% by volume or to a maximum of 660 kg/m$^3$ by weight. Most codes allow the gap width between bars and the thickness of concrete cover to be as small as the maximum size of aggregate plus 5 mm. If a 20 mm maximum size aggregate is used for the concrete mix, this will lead to a gap width to maximum size of aggregate ratio of smaller than 1.3 and it may then be advisable to limit the 10 - 20 mm aggregate to not more than 25% by volume.

*Segregation* is the separation of some of the ingredients, especially the coarse aggregate particles and the paste, from the bulk of the concrete mix due to free falling or sliding through surfaces during placing of the concrete, leading to non-uniform distribution of the various ingredients in the concrete. Nowadays, in order to speed up the concreting operation or to save the labour needed to properly compact the concrete mix or just for the purpose of ensuring better consolidation of the concrete mix, it is quite common to specify a relatively high workability of 150 mm slump or even higher. Such a concrete mix tends to contain a high water content. With an excessive amount of water added, the cohesiveness of the concrete mix will drop and the risk of segregation will increase (it should be noted that when the water content is low, the cohesiveness increases with the water content until the water content is equal to the water demand but when the water content is in excess of the water demand the cohesiveness decreases with the water content). To maintain a reasonable level of cohesiveness and increase the *segregation resistance* of the concrete mix, it is necessary to limit the water content of the concrete mix. Addition of more fine particles (such as fine aggregate, limestone filler, pulverized fuel ash and condensed silica fume) to the concrete mix can also help to increase the cohesiveness and improve the segregation resistance (finer particles stay with the bulk of the concrete mix much better than the coarser particles). Notwithstanding the use of a concrete with good segregation resistance, free falling and/or sliding over long distances should be avoided during placing of concrete.

Placing and compacting should be carried out under appropriate supervision. If the concreting operation is likely to take a long time, the workability retention of the concrete mix should be checked by carrying out slump tests of the concrete from time to time.

Concrete should be thoroughly compacted during placing by vibration or other means and worked into corners, gaps and any confined spaces in the mould carefully. Vibrations should be applied only until the expulsion of air has practically ceased. Over-vibration should be avoided as this may cause bleeding of the concrete mix and formation of a weak surface layer.

### 10.3.6 Curing

The code states that curing is the prevention of loss of moisture from the new concrete. This statement is not a complete description of curing. Curing is the maintenance of a suitable environment for the new concrete (1) to produce as much gel as possible so as to develop its full strength potential and reduce its permeability for better protection of the steel reinforcement from corrosion; (2) to avoid being damaged by plastic cracking or early thermal cracking; (3) to avoid being damaged by shock vibrations due to any nearby rock blasting or pile driving activities; and (4) to avoid being damaged by premature loading transmitted to it due to movement of the adjacent parts of the structure.

Hence, curing should comprise of at least the following:

- moisture control;
- thermal control;
- vibration control; and
- movement/deformation control.

*Moisture control* is particularly important if the water/cementitious ratio is low, the cement is of rapid hardening type and/or the concrete mix contains pozzolanic materials like pulverized fuel ash or condensed silica fume. It should start immediately after compaction of the concrete because premature drying out of the concrete mix due to solar radiation and wind could lead to plastic shrinkage cracking of the un-moulded surfaces of the freshly cast concrete (from the author's own experience, plastic shrinkage cracking is not uncommon in Hong Kong, especially in exposed areas where strong wind could cause rapid drying out of the freshly cast concrete). It should last at least 3 to 4 days until a substantial proportion of the chemical reactions for producing gel has been completed but for a concrete mix containing pozzolanic materials, it should last up to at least 4 to 6 days because pozzolanic reactions are generally slower than cement hydration (the minimum period of curing has been stipulated in Table 10.4 of the code). At the least, moisture control should include prevention of moisture loss by covering the freshly cast concrete with plastic sheets or impermeable membranes. The mould itself is a good protection against moisture loss and should therefore be kept in place for as long a time as practicable. Replenishment of water to keep the concrete saturated by mist spraying (also called mist curing) or water spraying (also called water curing) is in theory better than just prevention of water loss and should be considered for important structural elements, high strength concrete structures and high durability standard concrete structures. While the concrete is still fresh, mist curing should be applied but after the concrete has hardened both mist curing or water curing could be applied. However, mist/water curing should be considered in conjunction with thermal control and care should be taken to avoid generating a temperature shock to the concrete. The concrete could become quite hot during curing and application of cool mist/water onto the surfaces of the concrete could cause rapid contraction and almost immediate thermal cracking of the concrete. Intermittent wetting should also be avoided, as this would cause alternate swelling and shrinkage of the surface layer of concrete, thereby aggravating the surface cracking problem of the concrete.

*Thermal control* is particularly important if the concrete mix has a high cement content and/or a high cementitious materials content and/or the concrete structure to be cast is massive. The cement and cementitious materials contents of concrete mixes in Hong Kong tend to be on the high side partly because of the generous strength margins provided by the local producers to protect themselves from the risk of not meeting the strength test requirements and partly because of the generally high paste volumes of the concrete mixes provided to help avoid the formation of honeycombs. Because of the high cement and cementitious materials contents of the concrete mixes, large amounts of heat could be generated during the first few days after casting and the resulting temperature rise could be as much as 60$^{o}$C. The large temperature rise and the subsequent temperature drop could cause serious thermal cracking problems. The situation is worst if at the same time the structure to be cast is massive because in such case, it is difficult for the heat generated inside the core of the concrete structure to be dissipated (concrete is a bad heat conductor). The author has recommended in Section 7.2.4 of this handbook that if the cement content exceeds 450 kg/m$^3$ or the total cementitious materials content exceeds 550 kg/m$^3$, the contractor should be required to carry out an adiabatic curing test or a temperature rise evaluation test. Then, if the temperature rise of the concrete as measured by a true adiabatic curing test is higher than 45°C or as measured by an acceptable temperature rise evaluation test is higher than 40°C (a temperature rise evaluation test tends to yield a lower temperature rise because of heat loss), the contractor should be required to carry out thermal analysis and submit a proposal for thermal control of the curing concrete. The thermal control may comprise of external insulation, internal cooling or a combination of external insulation and internal cooling. Detailed design of the thermal control measures is beyond the scope of the code and specialist literature should be consulted when necessary.

*Vibration control* is not normally required unless there are rock blasting, pile driving or other vibration generating activities going on in the vicinity of the curing concrete. The Geotechnical Engineering Office had in 1994 engaged the Department of Civil Engineering, The University of Hong Kong to conduct an experimental study on the effects of blasting vibrations on green concrete (green concrete means curing concrete) and the results have been published in the following GEO report:
Kwan A.K.H. and Lee P.K.K., GEO Report No.102, A Study of the Effects of Blasting Vibration on Green Concrete, Geotechnical Engineering Office, Civil Engineering Department, The Government of HKSAR, 159pp.
For recommendations on vibration control, readers may refer to this report where a set of vibration control limits for typical concrete at different ages has been provided.

*Movement/deformation control* is also not normally required unless the adjacent parts of the structure could move during curing of the concrete. Nonetheless, this could be the most important measure for proper curing of the concrete in some cases. For instance, in precast segmental construction of a bridge, the stitching segment has to be cast in-situ. The concrete stitching segment could be damaged during curing if the construction traffic loads on the bridge cause movement of the deck and consequently deformation of the stitching segment. Hence, during curing of the stitching segment, no construction traffic should be allowed on the deck. If necessary, a temporary connection may be provided to hold the partially completed decks at the two ends of the stitching segment together so as to minimize the deformation of the stitching segment during curing. As another example, when an existing bridge is widened by constructing a new deck parallel to the old deck and connecting the two decks together with an in-situ concrete stitching slab, the concrete stitching slab could be damaged if the old deck is to remain open to traffic, which causes deflection of the old deck and consequently deformation of the stitching slab. Unfortunately, when an existing bridge needs widening, it is generally impracticable to close the old deck for any long period of time. Hence, traffic vibration from the old deck during curing of the concrete stitching slab is unavoidable. In such case, the provision of temporary shear connections holding the old and new decks together at certain points along the gap to be closed by the stitching slab could help to minimize the deformation of the stitching slab during curing. A general study on this problem and possible mitigating measures has been published in the following paper:
Kwan A.K.H. and Ng P.L., "Reducing damage to concrete stitches in bridge decks", Proceedings, Institution of Civil Engineers, Bridge Engineering, Vol.159, No.2, June, 2006, pp53-62.

### 10.3.7 Concreting in hot weather

At high ambient temperature, special precautions are necessary to avoid:

- reduction in working life of the fresh concrete due to shorter setting time arising from accelerated hydration;
- plastic shrinkage cracking due to loss of mix water arising from accelerated evaporation;
- early thermal cracking due to high temperature rise.

Considerations should be given to the following measures:

- use of retarder to prolong the setting time;
- protection of the exposed surfaces of the newly placed concrete from sunshine and strong wind to minimize evaporation;
- incorporation of supplementary cementitious materials such as pulverized fuel ash and ground granulated blastfurnace slag to reduce the rate of heat evolution and temperature rise during curing;
- reduction of the placing temperature by using ice instead of water as the mixing water and pre-cooling the aggregate before use.

The placing temperature of fresh concrete should not exceed 30$^{o}$C. However, where the risk of early thermal cracking is particularly high, it may be necessary to specify a lower placing temperature of say 25$^{o}$C. The ambient temperature should also be considered in specifying the placing temperature. The author suggests that the placing temperature should not exceed the ambient temperature plus 5$^{o}$C.

The concrete should be placed and compacted as soon as possible after mixing. To prevent moisture loss, curing of surfaces not protected by the moulds should commence immediately after compaction. The initial curing should include protective covering and, if the concrete mix is particularly dry, also mist curing. The protective covering should be made of impervious sheets, preferably pigmented to reflect radiation, supported away from the surface if the surface is not to be marked, and fastened at the edges to prevent air movement. Mist curing should be applied using a proper mist sprayer capable of generating mist in the form of minute droplets of water that are fine enough to stay in the air without dropping down.

### 10.3.8 Formwork and falsework

Formwork and falsework should be designed and constructed to safely withstand all the loads and vibrations that may occur during the construction process.

Formwork joints should be constructed to prevent loss of grout or mortar from the fresh concrete. The internal surfaces of the formwork must be clean. Release agents should be applied so as to provide a thin uniform coating to the forms without contaminating the reinforcement and the concrete should be placed while the releasing agents are still effective. Formwork spacers left in place should not impair the desired appearance or durability of the concrete structure.

Removal of formwork from the concrete should be executed without causing shock, disturbance or damage to the concrete. Removal of side formwork should be so arranged that the soffit form can be retained in position and properly supported on props until the concrete has achieved the required maturity.

The time for removal of formwork is determined by the following factors:

- whether the concrete structure has developed sufficient strength to support all the loads acting on it;
- whether the concrete has been sufficiently cured;
- avoidance of early thermal cracking; and
- the need for subsequent surface treatment.

The minimum formwork striking times for different types of structural elements cast of OPC concrete have been given in the code. However, from the curing point of view, the formwork should be left in place for as long as practicable. It should also be noted that if pulverized fuel ash has been added to the concrete, the formwork striking times may need to be extended for two reasons: (1) the early strength development of concrete containing pulverized fuel ash is slower; and (2) concrete containing pulverized fuel ash needs a longer period of curing.

In order to avoid generating temperature shock, which may cause early thermal cracking, the formworks for massive concrete structures (such as pile caps, big columns, thick walls and large portal beams etc, of which the minimum dimension is larger than 1.0 m) should not be removed until the temperature of the concrete has reached the peak and dropped to near the ambient temperature. For the same reason, after removal of the formworks, cool water should not be sprayed onto the hot concrete surfaces or otherwise thermal cracks might appear immediately.

#### 10.3.9 Surface finish

The formwork should be designed and constructed in such a way that there is no unacceptable loss of fines, blemish or staining of the concrete surfaces. Blow holes up to 10 mm may be expected but the concrete surface should otherwise be free from voids, honeycombs or other large defects.

Where a particular finish is required for appearance, the requirements should be specified directly or by reference to sample surfaces. Exposed smooth off-the-form and board marked finishes are only for interior use. Trial casting and mock up should be carried out before the formworks and surface preparation procedures are accepted.

The surface quality of concrete is dependent on the constituents of the mix and their proportions, the mixing, handling, compaction and curing procedures, and the type of formwork and release agent used. To maintain a consistent surface finish, the same concrete mix from the same batching plant using the same sources of constituents, the same type of formwork and release agent and the same gang of workers should be employed throughout.

### 10.3.10 Construction joints

Construction joints should be kept to the minimum number necessary. Their locations should be decided and agreed before the concrete is placed and they should normally run at right angles to the axis of the member.

The concrete at a vertical construction joint should be formed against a stop end and bonded with the concrete subsequently placed against it, without provision for movement.

The top surface of concrete to become a horizontal construction joint should be level and reasonably flat. If a kicker is provided, it should be at least 70 mm high and carefully constructed.

For a joint to transfer tensile or shear stresses, the surface of the first pour should be roughened to increase the bond strength and to provide aggregate interlock. The surfaces of horizontal joints can be roughened without disturbing the coarse aggregate particles by spraying the joint surfaces approximately 2 to 4 hours after the concrete is placed with a fine spray of water and/or brushing with a stiff brush. Vertical joints can be treated similarly if a retarder is used on the stop end to allow the joint surface to be treated after removal of the stop end. If permitted, mesh or expanded metal stop ends, which should not extend into the cover zone, can provide a rough face to the joint, which can also be sprayed whilst the concrete is still green.

Where it is not possible to roughen the joint surface until the concrete has hardened, the coarse aggregate near the surface should be exposed by sand blasting or by scale hammer or other mechanical devices. Powerful hammers that may dislodge the coarse aggregate particles should not be used.

The joint surface must be clean and free from loose particles before the fresh concrete is placed. Slight wetting may be needed prior to the new concrete being placed to prevent loss of mix water. New concrete placed close to the joint must have an adequate fines content.

### 10.3.11 Movement joints

The movement joints must have sufficient gap widths to allow future movements at the joints.

Joint filler forming the gap should be firmly fixed to the first placed concrete. If more than one strip is used, the ends should be butted closely together and taped to prevent grout leakage, which may reduce the effective gap width of the joint.

Flexible water stops should be fixed so that they cannot become displaced from their intended positions and so that the concrete surrounding them can be fully compacted.

Contraction joints may alternatively be introduced by the use of crack inducers.

## 10.4 Reinforcement

### 10.4.1 General

Reinforcement should conform to Hong Kong Construction Standard CS2: Carbon Steel Bars for the Reinforcement of Concrete and the acceptable standards listed in Annex A of the code.

### 10.4.2 Cutting and bending

Reinforcement should be cut in accordance with the acceptable standards. There could be many different ways of cutting the required lengths of bars from standard lengths (normally 12 or 18 m) but the amount of leftover (short lengths remaining after cutting that cannot be used as reinforcement) varies with the method of cutting. Hence, it may be advisable to optimise the cutting schedule to minimize the leftover; commercial software is now available for such purpose.

Reinforcement should be bent in accordance with the acceptable standards. Bending should be carried out using mandrels such that each bend has a constant curvature. Grade 250 reinforcement bars projecting from concrete may be bent, re-bent or straightened provided the internal radius of bend is not less than that specified in the acceptable standards. Grade 500 reinforcement bars projecting from concrete should not be bent, re-bent or straightened without the engineer’s approval. After bending, re-bending, straightening or any kind of reshaping, each bar should be inspected for signs of fracture.

### 10.4.3 Fixing

The reinforcement should be firmly tied together and securely fixed in position relative to the formwork to maintain the bars in their prescribed positions during concreting. Nominal concrete cover should be specified to all steel reinforcement including links. The specified nominal cover should be maintained by the use of approved spacers and chairs and the actual cover provided should not be less than the nominal cover minus 5 mm. The position of reinforcement should be checked before and during concreting to ensure that the nominal cover is maintained within the prescribed limits. Where considered appropriate, a cover meter may be used to check the cover to reinforcement in hardened concrete.

### 10.4.4 Surface condition

The surfaces of reinforcing bars should be clean and free from deleterious substances such as oil, grease, mud, loose rust or mill scale that may adversely affect the bond between the bars and the concrete. If necessary, the steel surfaces should be cleaned by brushing and/or washing before the reinforcing bars are fixed. The surface condition of the reinforcing bars should be examined again prior to concreting.

10.4.5 Laps and joints

Laps and joints should be made in accordance with the specifications and the details shown in the drawings, or as agreed with the engineer.

10.4.6 Welding

Generally, welding should be carried out under controlled conditions in a factory or workshop; welding on site should be avoided. Welding should be carried out only on reinforcing steel that has the required welding properties and should not generally be applied at or near bends. All welding should be carried out in accordance with the acceptable standards by qualified welders, whose competence should be demonstrated prior to and periodically during the welding operations.

Welding may be used for the following purposes:
- non-structural spot welds for fixing steel bars in position;
- structural welds for transferring loads between steel bars or between steel bars and other steel members.

The types of welding permitted include metal-arc welding, flash butt welding and electric resistance welding. Other methods of welding may also be used subjected to approval of the engineer and the reinforcement manufacturer.

Welded joints between parallel bars of principal tension reinforcement should be staggered in the longitudinal direction, unless otherwise permitted by the engineer. Welded joints may be considered as staggered if the longitudinal distance between them is not less than the end anchorage length for the bars. The definition of “end anchorage length” shall refer to Clause 8.4.6 and Clause 8.7.2. According to Clause 8.4.6, the anchorage length shall be min. bending radius plus 4 · □□□. · □ total 8 $\phi$.

Welds for welded lapped joints should have for each single pass a length of run not exceeding 5 times the diameter of the bar. If a longer length of run is required, it should be divided into sections with the space between runs not less than 5 times the diameter of the bar.

## 10.5 Prestressing steel

10.5.1 General

Prestressing tendons should conform to the acceptable standards listed in Annex A of the code. The tendons, anchorages, couplers and sheaths should be those specified in the design and should be identifiable as such.

10.5.2 Transport and storage

The tendons, anchorages, couplers and sheaths should be protected from harmful influences during transport, storage and handling.

To ensure protection of the tendons, the following should be avoided:

- mechanical deformation or heating during handling;
- exposure to rain or contact with the ground;
- welding or cutting of steel in the vicinity of tendons without protecting the tendons from splashes;
- after manufacture, any welding, on-site heat treatment or metallic coating such as galvanizing; and
- contamination likely to affect the durability or bond of the tendons.

## 10.5.3 Fabrication

At the time of incorporation in the structural member, all prestressing tendons, sheaths or ducts should be free from harmful matters such as loose rust, oil, paint or other lubricants. Oiled or greased tendons may be used under certain circumstances, if agreed among the parties involved. Tendons may be cleaned by wire brushing.

Low and normal relaxation wires and prestressing strands should be transported in coils of sufficiently large diameter to ensure that they run off straight from the coils. Prestressing bars should be straight; small adjustment for straightness may be carried out on site by hand in cold under the supervision of the engineer but bars bent in the threaded portion should be rejected.

Cutting to length and trimming of ends should be by either high-speed abrasive cutting or oxy-acetylene flame cutting. Neither the flame nor splashes should come into contact with the anchorage or other tendons. Post-tensioned tendons should not be cut less than one diameter from the anchor and the temperature of the tendon adjacent to the anchor should not exceed 200°C.

## 10.5.4 Placing

The tendons, couplers, anchorages and sheaths should be accurately placed in the positions shown on the drawings. The permitted deviation in the locations of the tendons, sheaths or duct formers should be $\pm 5$ mm unless otherwise stated. The tendons, sheaths or duct formers should be securely fixed such that they will not be displaced by disturbance from nearby activities or during concreting.

Sheaths and extractable cores should be strong enough to retain their correct section and profile. Joints should be the minimum practicable and adequately sealed to prevent ingress of any material until grouting has been completed. Ends of ducts should be sealed and protected after stressing and grouting. Extractable cores should not be coated with release agent except with the approval of the engineer and should be retained until the concrete has sufficient strength to prevent damage. Joints in adjacent sheaths should be staggered at a minimum of 300 mm apart. Damage to ducts can occur during concreting, and if the tendon is to be inserted later, the ducts should be dollied during concreting to ensure a clear passage for the tendon.

### 10.5.5 Tensioning

*Tensioning apparatus and procedures:*

The tensioning apparatus and procedures should be in accordance with:

- The tendons should be safely and securely attached to the tensioning device.
- Where wires or strands are stressed simultaneously, they should be of approximately equal length between anchorage points.
- The tensioning apparatus should gradually impose a controlled force and not induce dangerous secondary stresses in the tendons, anchorages or concrete.
- The tendon force should be measured by a direct reading load cell or obtained indirectly from a pressure gauge in the jack. The extension of the tendon and any movement of the tendon in the gripping device should be measured. The load cell or pressure gauge should be accurate to within 2%. The tendon elongation should be measured to within 2% or 2 mm whichever is less.
- The tensioning apparatus should have been calibrated within six months.

*Pre-tensioning:*

The tension should be fully maintained during the period between tensioning and transfer. The transfer should take place slowly to minimize shock that would adversely affect bond between the tendons and the concrete. For straight tendons used in the long-line method, locator plates should be distributed throughout the length of the casting bed to ensure proper positioning of the tendons during concreting. To permit transfer of prestressing force to the concrete in each unit on the long line, each unit should be free to slide in the direction of the line. For straight tendons in the individual mould system, the moulds should provide the reaction to the prestressing force without distortion, For deflected tendons, to minimize frictional loss, the deflectors should allow free movement of the tendons. The deflector should have a minimum radius of 5 times the tendon diameter for wire or 10 times the tendon diameter for strand and the total angle of deflection should not exceed $15^{o}$.

*Post-tensioning:*

The anchorage system comprises of the anchorage itself together with the tendons and the reinforcement designed to act with the anchorage. The anchorage and the design of the entire anchorage system should conform to the acceptable standards. Where proprietary anchorages are used, the anchoring procedure should strictly follow the manufacturer's instructions and recommendations. Split wedge and barrel-type anchors should be of such form that under the loads imposed during tensioning, the wedges do not reach the limit of their travel before developing sufficient lateral force to grip the tendons. Allowance for draw-in of the tendons during anchoring should be in accordance with the engineer's instructions and the actual slip occurring for each individual anchorage should be recorded. After the tensions have been anchored, to avoid shock, the force exerted by the tensioning apparatus should be decreased gradually and steadily. The deflector should have a minimum radius of 50 times the tendon diameter and the total angle of deflection should not exceed 15$^{o}$. Where the radius of the deflector is less than 50 times the tendon diameter or the total angle of deflection exceeds 15$^{o}$, the loss of strength of the tendon should be determined by test and allowed for. Before tensioning, the tendons should be shown to be free to move in the ducts. During stressing, allowance should be made for the friction in the jack, unless load cells are used. Measurement of the tendon extension should not commence until any slack in the tendon has been taken up. Tensioning should continue until the required tendon extension and/or load is achieved. The required tension should allow for any draw-in of the tendon at the non-jacking end. A check on the accuracy of the frictional loss assumed at the design stage is provided by comparing the measured tendon force with that calculated from the extension; if the difference is greater than 6%, corrective measures should be taken with the approval of the engineer. Records should be kept of all tensioning operations, including the measured extensions, pressure gauge or load cell readings and draw-in at each anchorage.

### 10.5.6 Protection and bond of prestressing tendons

*General:*
Prestressing tendons must be protected against mechanical damage and corrosion. Protection against fire damage may also be required. Stressed tendons are normally required to be bonded to the structure after all the tensioning and anchoring operations are completed.

*Protection and bond of internal tendons:*
Cement grout or cement sand grout may be employed to protect and bond internal tendons to the structure.

*Protection and bond of external tendons:*
External tendons should generally be protected against mechanical damage and corrosion by an encasement of dense concrete or dense mortar of adequate thickness. Consideration should be given to the differential movements that may arise between the structure and the applied protection. If the applied protection is concrete or mortar and there is the possibility of undesirable cracking, a primary corrosion protection that will not be impaired by the differential movements should be used. Where the external tendons are required to be bonded to the structure, the concrete encasement should be suitably reinforced and tied to the structure.

### 10.5.7 Grouting

*General:*
The main objectives of grouting ducts in post-tensioned concrete members are:

- to prevent corrosion of the tendons; and
- to allow effective transfer of bond stresses.

To meet the first objective, the grout should be alkaline and dense and should cover the tendons. It also should not contain any deleterious materials. To meet the second objective, the grout should completely fill all voids in the ducts and when hardened should have the required bond strength to allow development of effective bond between the tendons and the sides of the ducts.

*Duct design and construction:*
Sudden changes in the diameter or alignment of ducts should be avoided. Vents should be provided at crests in the duct profile, at any major changes in the section of the duct, and elsewhere if required. Vents should be provided at high points if the difference in level between them and low points is greater than 0.5 m. Vents should also be provided at anchorages. All vents should be closable. Vents to be used as entry points should be threaded to permit the use of a screwed connector from the grout pump. Vents and injection connections to the ducts should be secure and tight. They should be able to withstand disturbance before concreting and the pressure generated during grouting. Lined ducts should be kept dry before grouting to prevent corrosion of the tendons or having excess water in the grout; it may be necessary to blow dry oil-free air through a lined duct occasionally to prevent condensation if it is left ungrouted for a considerable time. Unlined ducts may have to be wetted before grouting to prevent absorption of water from the grout by the surrounding concrete; it may be preferable to flush the ducts using clean water with compressed air. Vertical ducts should be sealed at all times before grouting to prevent ingress of rain and debris.

*Properties of grout:*
The grout should have high fluidity (same as flowability) and high cohesiveness when plastic and low shrinkage and adequate strength when hardened. The fluidity may be assessed using the immersion or the cone test methods set out in the acceptable standards. The cohesiveness is a measure of the resistance to segregation, bleeding and sedimentation. Cohesiveness may be increased by reducing the water/cementitious ratio, adding pulverized fuel ash and/or a viscosity modifying admixture. The 100 mm cube strength of the grout should not be less than 27 N/mm$^2$ at 28 days.

*Composition of grout:*
The grout should be composed of ordinary Portland cement and water but may also contain sand, filler and admixtures. Normally, sand is included only in grouts to be placed in ducts with a diameter of more than 150 mm. Pulverized fuel ash may also be added as filler if considered suitable. Although pulverized fuel ash is often regarded just as filler in the grout, it actually also functions as a supplementary cementitious material and improves the cohesiveness of the grout. Plasticizing and viscosity modifying admixtures may be added to improve the fluidity and cohesiveness of the grout. However, the use of gas generating admixtures needs very careful consideration. The generation of an excessive amount of gas in the grout could substantially weaken the grout and therefore if the gas generation is not properly controlled, the grout could be seriously damaged. The author has throughout the years seen quite a number of failure cases due to improper use of gas generating admixtures. Extreme care should be taken while using any gas generating admixture and trial mixing with different dosages of the admixture added to the grout should be carried out to evaluate the possible effects of over-dosage before use.

*Batching and mixing of grout:*
The water/cement ratio should not exceed 0.44. For a neat cement grout, the optimum water/cement ratio is probably around 0.40. With a suitable plasticizing

admixture added, the water/cement ratio may be reduced to 0.35. The quantity of sand or filler used should not exceed 30% of the mass of the cement. The grout should be mixed in a machine capable of producing a homogeneous colloidal grout and keeping the grout in slow continuous agitation until it is ready to be pumped into the ducts. Water should be added to the mixer first, followed by the cement. When these two materials are thoroughly mixed, sand or filler may be added. The minimum time of mixing will depend upon the type of mixer and the manufacturer's recommendations should be followed. Generally, the minimum mixing time is around 0.5 to 2.0 minutes. Grout mixing should not normally be continued for more than 4 minutes. Where admixtures are used, the manufacturer's recommendations should be followed. There is, however, the major difficulty of exercising proper quality control. Grout batching and mixing is usually carried out on site using a mini grout mixer. The batching accuracy cannot be compared to modern computer controlled batching plants that we are using to produce ready mixed concrete. Hence, the quality control of grout production is generally not as good as that of ready mixed concrete production. Very close site supervision is also needed. During grouting, the workers are usually pressed for timely completion of the works and under such situation the quality of works may no longer be the first priority.

*Grouting procedure:*
The grouting procedure should be in accordance with the acceptable standards.

*Some other general recommendations:*
The Concrete Society Technical Report No.47: Durable Bonded Post-tensioned Concrete Bridges has made the following recommendations:

- Ducts and vents to be pressure tested.
- Vents to be positioned at low points, high points and beyond high points in the direction of grout flow.
- Tighter grout specification to be provided covering fluidity, bleeding, sedimentation, volume change, strength and sieve tests.
- Use of a plasticizer in the grout.
- Additional grout testing.
- Requirements for experience and training of operatives and supervisors.

In Hong Kong, full-scale grout trials are not normally required although grout trials are specified in the Civil Engineering Department General Specifications. The details are left to the engineer. The current practice could continue and full-scale trials are specified only if considered necessary, for example in critical structures, to demonstrate there can be adequate protection to the tendons. The use of electrically non-conductive ducts, which can help to mitigate the effects of de-icing salts, has been recommended in the UK. Currently in Hong Kong, both steel and HDPE ducts are used and allowed. Nevertheless, in Hong Kong, with much of the infrastructure either over seawater or near to the coast, there could be a supply of salt from the sea and in the air. The electrically non-conductive duct, together with the concrete cover and grout, will provide a multi-layer protection. This should be considered and reviewed with particular reference to any data and corrosion problems of prestressing ducts in Hong Kong. Designed grouts are not common in Hong Kong, where cement grout is invariably used. Again the use of designed grouts should be considered and reviewed with particular reference to any data and problems associated with cement grouting in Hong Kong.

# 11 QUALITY ASSURANCE AND QUALITY CONTROL

## 11.1 Scope

The scope covers the necessary control measures for the design and construction of concrete structures, which should comprise of essential actions and decisions in accordance with approval procedures, as well as checks to be made, to ensure that all specified requirements are met.

## 11.2 Quality assurance

A quality assurance system should be established to ensure that the various control measures are enforced and that the necessary corrective actions are made until all specified requirements are met. The quality assurance system should include a check list of all verifications to be made at different stages of design and construction, proper documentation of the verification results, identification of necessary corrective actions, approval procedures and records, and most important of all, a clear distinction of the roles and responsibilities of all parties involved in the design and construction project.

Though some people may disagree, the author is of the view that for a quality assurance scheme to really work in this commercial world, there needs incentive for good quality and penalty for bad quality; otherwise everything will be empty talk and the whole scheme will become just for the purpose of looking for a scapegoat when things go wrong. There is in reality a cost to pay for better quality. Without reasonable remuneration, it is simply unrealistic to expect high quality production. The working culture is also important. If the workers involved in the project can have their contributions better recognized by the society, the workers will feel proud of what they are working on and strive to produce a high quality product.

## 11.3 Classification of the control measures

There are three basic control systems: (1) internal control; (2) external control; and (3) conformity control.

*Internal control:*
Internal control is carried out by the designer, the contractor, the supplier or any party responsible for part of the works, each within his/her specific scope. It is exercised either on his/her own “internal” initiative or according to “external” requirements stipulated by the client or by an independent organization.

*External control:*

External control is carried out by an independent organization charged with this task by the client or by the relevant authority. External control may consist of the verification of internal control measures that are required by external requirements or additional checking procedures independent of the internal control system.

*Conformity control:*
Conformity control is exercised to verify that a particular service or production function has been carried out in conformity with the specification previously established. Conformity control is generally part of external control.

## 11.4 Verification systems

The frequency and intensity of control depend on the chances and consequences of non-compliance with the specification requirements in the various stages of design and construction. Different control measures are combined in a verification system to ensure the effectiveness of the control system.

## 11.5 Control of each stage of design and construction process

According to the purpose and time of the control, the following stages may be distinguished:

- control of the design;
- control of the production and construction; and
- control of the completed structure.

## 11.6 Control of design

Control of design shall conform to the statutory and administration procedures. It should be noted that despite the approval given by the relevant authority, it is ultimately the responsibility of the designer to ensure that the design assumptions are correct, the design calculations contain no mistakes, the computer programs used are suitable for the type of structure being designed and have been thoroughly verified, the drawings are accurate presented, the specifications are proper and the completed structure will perform reasonably well to meet with the requirements of Section 2.1 of the code. It is up to the designer to arrange internal control to ensure that the design is of an acceptable standard.

## 11.7 Control of production and construction

*Objectives:*
The control of production and construction comprises of all measures (e.g. inspections and tests) necessary to maintain and regulate the quality of materials and standard of the workmanship in conformity with the specified requirements.

*Production of concrete:*
Structural concrete for all works should be obtained from concrete suppliers who are certified under the Quality Scheme for the Production and Supply of Concrete (QSPSC), except for projects located at remote areas or where the volume of concrete involved is less than 50 $m^3$. Even for these "exceptional" projects, the structural concrete should be obtained from a supplier operating an approved quality system.

*Items and elements of production and construction:*
The items which need production and construction control are summarized in Table 11.1 of the code. The production and construction control should include:

- initial tests and checking procedures;
- tests and checks in the course of construction; and
- final tests and checks.

Different verification systems may be appropriate for:

- a continuous production: the aim is to achieve a uniform quality of the products in the long term; and
- a single product: the aim is to comply with the specification requirements.

For a single product, it may be appropriate to concentrate on precautionary measures, in particular on initial tests and on checks during construction.

*Initial tests:*
Initial tests should be carried out before the start of the construction in order to check that the materials and equipment to be used are of the required standards and that the construction methods to be used are feasible and have no detrimental effects on the performance of the completed structure.

*Checks during construction:*
The dimensions and the properties of the materials and components built into the structure should be subjected to verification during construction. The results of the verification measurements/tests should be properly filed for assessment by all parties concerned. For ready mixed concrete, the delivery note should be in accordance with QSPSC. For reinforcing steel, the delivery ticket should include information on the origin and the identity of the steel delivered and the steel should have labels and rolling marks for identification.

*Conformity controls:*
Conformity control is the combination of actions and decisions to be taken in order to verify that all the requirements, criteria and conditions laid down in the specifications have been complied with. This implies completing relevant documentation and granting approval for acceptance and use in the construction.

For the conformity control of concrete, the sampling frequencies and acceptance criteria set out in Clause 10.3.4 of the code apply. For the conformity control of steel, Clause 3.2.1 of the code applies.

*Control and maintenance of the completed structure:*
Access for control and maintenance of the completed structure should be provided. This may, in actual practice, be rather difficult, especially for the foundation or any parts of the structure that are buried underground or permanently covered up. Where access cannot be provided, considerations should be given to the possibility of installing sensors to monitor the corrosion, deformation and vibration of the structure.

# 12 PRESTRESSED CONCRETE

## 12.1 Basis of design

In the design of prestressed concrete structures, it is difficult to predict whether the ultimate limit state or the serviceability limit state will be the critical one because the serviceability limit state requirements for prestressed concrete structures are generally more stringent than those for ordinary reinforced concrete structures. Hence, the usual practice in the design of reinforced concrete structures of first designing the structure in accordance with the ultimate limit state requirements and then checking whether the structure so designed satisfies the serviceability limit state requirements based on the assumption that the ultimate limit state will be the critical one may not be applicable to prestressed concrete structures. In the design of prestressed concrete structures, it is quite often that the serviceability limit state is the more critical one. In fact, it may even be the stress condition at the transfer stage that governs the design.

Alternative design methods based on more realistic design assumptions than those made in the code may be adopted if considered as more appropriate, especially for the design of uncommon structures with special features.

In the assessment of the likely behaviour of a prestressed concrete structure or member, the amount of flexural tensile stresses allowed under service load is dependent on the serviceability classification of the structure or member. There are three serviceability classes:

- Class 1: no flexural tensile stresses.
- Class 2: flexural tensile stresses allowed but no *visible* cracking.
- Class 3: flexural tensile stresses allowed but subjected to a maximum surface crack width of 0.1 mm for members in exposure conditions 3 or 4 and a maximum surface crack width of 0.2 mm for all other members.

The definition for Class 2 given in Clause 12.1 of the code is rather vague and ambiguous because it is not known what is meant by "no *visible* cracking". Whether a crack is visible or not is dependent not only on the crack width, but also on the lighting condition and the eyesight of the observer. In fact, Clause 12.3.4 of the code redefines Class 2 as "flexural tensile stresses allowed but the design tensile stresses should not exceed for pre-tensioned members, the design flexural tensile strength of the concrete and for post-tensioned members, 0.8 of the design flexural tensile strength of the concrete". Hence, the real meaning of "no visible cracking" is "no potential cracking". Readers should bear this in mind and always refer to Clause 12.3.4 for serviceability classification.

Comparing the above crack width limitations for Class 3 prestressed concrete structures with those stipulated for prestressed members with bonded tendons in Table 7.1, it can be seen that the above limitations are more stringent than those stipulated in Table 7.1. Based on the spirit that if there is any inconsistency in requirements, the more stringent requirement should apply, the crack width limitations set in this chapter should override those given in Chapter 7.

*Design loads:*
The design loads to be used for the ultimate limit states are those given in Clause 2.3.2 of the code, i.e. are to be taken as the characteristic loads multiplied by the respective load safety factors depending on the load combination being considered. On the other hand, the design loads to be used for the serviceability limit states are to be taken as the characteristic loads. However, Sections 7.1 and 7.2 of the code suggest that for prestressed members with bonded tendons, the frequent load combination (i.e. the expected high load under normal condition) may be used to check crack width limits and the quasi-permanent load combination (i.e. the expected average load under normal condition) may be used to check deflection limits. Again, based on the spirit that if there is any inconsistency in requirements, the more stringent requirement should apply, the characteristic loads as required in this chapter should be used for the serviceability limit states.

*Design strengths:*
The minimum grades of concrete for post-tensioning and pre-tensioning are C35 and C40, respectively. The concrete strength at transfer should be not less than 25 N/mm$^2$. The specified characteristic strengths of steel reinforcement and prestressing tendons are given in Clause 3.2 and Clause 3.3, respectively, of the code.

## 12.2 Structures and structural frames

*Analysis of structures:*
Complete structures and complete structural frames may be analysed using the performance based approach in accordance with Clause 2.6.3 or following the recommendations in Chapter 5 of the code. Considerations should be given to the effects of the construction sequence. Individual members may be analysed as per Section 12.3 of the code.

*Relative stiffness:*
The relative stiffness should generally be based on the concrete section as described in Clause 5.1.2 of the code.

*Redistribution of moments:*
For concrete of strength grade not exceeding C70, redistribution of moments may be applied, provided the following conditions are satisfied:

Condition 1. Equilibrium between internal and external forces is maintained.

Condition 2. The reduction made to the maximum design moment derived from elastic analysis does not exceed 20%.

Condition 3. Where the design moment is reduced at a section, the neutral axis depth $x$ should be checked to see that it is not greater than the values specified in Equations 6.4, 6.5 and 6.6.

## 12.3 Beams

### 12.3.1 General

The geometric properties of prestressed concrete beams are defined and limited in the same ways as for reinforced concrete beams in Clause 6.1.2.1 of the code, except that the overall depth of the member should be used instead of the effective depth.

### 12.3.2 Slender beams

Clause 6.1.2.1 should be referred to regarding the slenderness limits for avoiding lateral instability. Particular attention should be paid to possible instability of the beams during construction as well as when the beams are in their final positions.

### 12.3.3 Continuous beams

Continuous beams may be analysed using an elastic analysis method with the following arrangements of loads considered: (1) two adjacent spans loaded with maximum design loads and all other spans loaded with minimum design loads; (2) alternate spans loaded with maximum design loads and all other spans loaded with minimum design loads; and (3) all spans loaded with maximum design loads. Redistribution of the moments obtained by elastic analysis may then be carried out as per Clause 12.2.3.

### 12.3.4 Serviceability limit state for beams

*Section analysis:*
For analysis of sections at serviceability limit state, it may be assumed that plane sections remain plane and that the materials are elastic and linear. In general, only the load arrangements at the following two stages: (1) immediately after the transfer of prestress, and (2) after all losses of prestress have occurred, need to be considered. For such analysis, the effects of dead and imposed loads on the forces in the tendons may be ignored.

*Compressive stresses in concrete:*
In flexure, the compressive stress should not exceed $0.33f_{cu}$ except within the range of support moments in continuous beams and other statically indeterminate

structures where the compressive stress may be increased to $0.4f_{cu}$. In direct compression, the compressive stress should not exceed $0.25f_{cu}$.

*Flexural tensile stresses in concrete:*
At mortar or concrete joints of precast units, no tension should be allowed. Elsewhere, the tensile stresses should not exceed the following limits for different classes of prestressed concrete structures/members:

- 
- Class 1: No flexural tensile stress.
- Class 2: The design flexural tensile stress should not exceed for pre-tensioned members, the design flexural tensile strength of the concrete and for post-tensioned members, 0.8 of the design flexural tensile strength of the concrete (the design flexural tensile strength of concrete may be taken as $0.45\sqrt{f_{cu}}$).
- Class 3: Although cracking is allowed, it is assumed that the concrete section is uncracked and that design hypothetical tensile stresses exist at the limiting crack widths (the design hypothetical tensile stresses are given in Table 12.2 for depth = 400 mm and are to be modified by the coefficients in Table 12.3 for depth ≠ 400 mm). When additional reinforcement is contained within the tension zone, and is positioned close to the tension faces of the concrete, these modified design hypothetical tensile stresses may be increased by an amount that is in proportion to the cross-sectional area of the additional reinforcement (expressed as a percentage of the cross-sectional area of the concrete in the tension zone), depending on which group the member is in (see Table 12.2 for the definitions of the different groups). For 1 % of additional reinforcement, the stresses may be increased by 4.0 N/mm$^2$ for members in groups a) and b), and by 3.0 N/mm$^2$ for members in group c). For other percentages of additional reinforcement, the stresses may be increased in proportion up to a limit of $0.25f_{cu}$. When a significant proportion of the design service load is transitory so that the whole section is in compression under the frequent load combination (dead load plus frequently occurring imposed load; see Section 7.2 of this handbook and note that this load combination of dead load plus frequently occurring imposed load is not a permanent load), the foregoing hypothetical tensile stresses may be exceeded under the full service loads (the full characteristic loads).

### 12.3.5 Stress limitations at transfer for beams

*Compressive stresses in concrete:*
The compressive stresses should not exceed $0.5f_{ci}$ at the extreme fibre nor $0.4f_{ci}$ for near uniform distributions of prestress, where $f_{ci}$ is the concrete strength at transfer.

*Flexural tensile stresses in concrete:*
The tensile stresses should not exceed the following limits:

- Class 1: 1.0 N/mm$^2$.
- Class 2: for pre-tensioned members $0.45\sqrt{f_{ci}}$ and for post-tensioned members $0.36\sqrt{f_{ci}}$.
- Class 3: same as for Class 2; if the tensile stress limit is exceeded, the section should be considered as cracked.

12.3.6 Deflection of beams

There are no deemed-to-satisfy rules. When it is considered necessary to calculate the deflection, the following methods may be used:

- Class 1: Elastic analysis based on the concrete section.
- Class 2: Elastic analysis based on the concrete section.
- Class 3: If the frequent load combination results in stresses not greater than those in Table 12.1, an elastic analysis based on the concrete section may be used. Otherwise, rigorous analysis based on the moment-curvature relationship for cracked section should be carried out.

12.3.7 Ultimate limit state for beams in flexure

*Section analysis:*
The following assumptions should be made:

- Plane sections remain plane.
- The design stresses in the concrete in compression are either derived from the stress-strain curve given in Figure 3.8 of the code with $\gamma_m = 1.5$ or taken as $0.45f_{cu}$ over a depth (from the compression face) equal to 0.9 times the depth of the compression zone.
- The tensile strength of concrete is negligible.
- The design stresses in any additional reinforcement and in bonded tendons, whether initially tensioned or untensioned, are derived from the appropriate stress-strain curves as given in Figure 3.9 and Figure 3.10, respectively.
- The design stress in unbonded tendons is limited to the values given by Equation 12.2 unless more rigorous analysis justifies a higher value.

*Design formulae:*
The resistance moment $M_u$ of a beam containing bonded or unbonded tendons, all of which in the tension zone, may be obtained as:

$$M_u = f_{pb}A_{ps}(d - d_n) \qquad \text{Equation 12.1}$$

For a rectangular beam or a flanged beam with flange thickness not less than 0.9*x*, $d_n$ may be taken as 0.45*x*. For bonded tendons, $f_{pb}$ and *x* may be obtained from Table 12.4 of the code. For unbonded tendons, $f_{pb}$ and *x* may be obtained from Equation 12.2 and Equation 12.3 respectively of the code. Table 12.4, Equation 12.2 and Equation 12.3 of the code are the same as those given in BS8110: Part 1: 1997, except that the design strength of the tendons is taken as $0.87f_{pu}$ instead of $0.87f_{pu}$ as in BS8110.

*Allowance for additional reinforcement in the tension zone:*
The additional reinforcement in the tension zone may be taken as equivalent to prestressing tendons with an area of $A_s f_y / f_{pu}$.

Worked Example 12.1: Check of ultimate moment of resistance for a pre-tensioned beam

A precast concrete double T-beam, as shown in Figure 12.1, carries an imposed load of 4.5kN/m$^2$ over a span of 12.5m. **The beam is pretensioned with six cable each consists of one 15.7mm diameter super strand**, each being stressed initially to 70% of the characteristic strength.
Check the ultimate moment of resistance of the mid-span section and provide untensioned reinforcement if necessary.

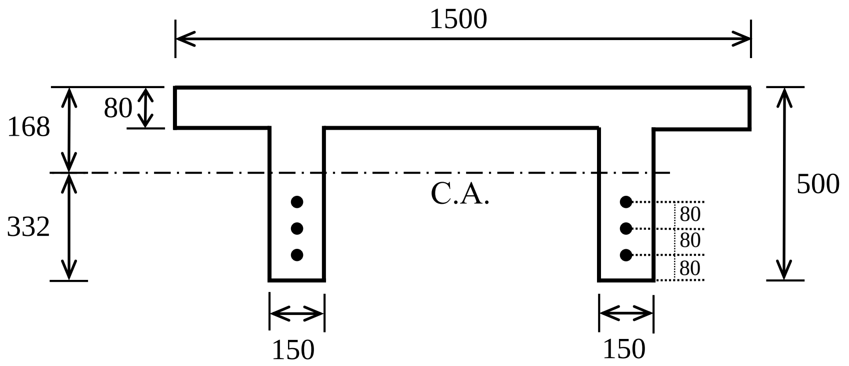



Figure 12.1 Worked example – precast pretensioned RC beam

Properties of beam:
A = 246000 mm$^2$
I = 5.758 × 10$^9$ mm$^4$

Concrete characteristic strength: 50 N/mm$^2$

Eccentricity of the tendon, $e$ = 332 – 160 = 172mm
Initial prestressing force $P_i = 265 \times 6 \times 70\% = 1113$kN
Prestress at transfer $P_t = 1113 \times (1 - 15\%) = 946.1$kN
Loss Ratio $R_0 = \frac{1-25\%}{1-15\%} = 0.882$

Nominal steel area of 15.7mm diameter super strand: 150mm$^2$
Gross area of tendons = 900mm$^2$

Dead load $\text{DL} = 24 \times 246000 \times 10^{-6} = 5.904\text{kN/m}$
Live load $\text{LL} = 4.5\text{kN/m}^2 \times 1.5 = 6.75\text{kN/m}$

At ultimate limit state,
1.4DL + 1.6LL = 1.4(5.904) + 1.6(6.75) = 19.07 kN/m

Ultimate bending moment, $M_u^R = \frac{19.07 \times 12.5^2}{8} = 372$kN-m at mid span

$M_u = f_{pb} A_{ps}(d - d_n)$ Equation 12.1

$f_{pu} A_{ps} = 265 \times 6 = 1590$ kN

$f_{pu} = {1590 \times 10^3}/{900} = 1767\ \text{N/mm}^2$

$d$ = 168 + 172 = 340mm

$b$ = 1500mm

$\frac{f_{pu} A_{ps}}{f_{cu} bd} = \frac{1590 \times 10^3}{50 \times 340 \times 1500} = 0.0624$

$f_{pe}$ = (0.7)(1-0.25)$f_{pu}$ = 0.525$f_{pu}$

${f_{pe}}/{f_{pu}} = 0.525$

| | | ${f_{pe}}/{f_{pu}}$ | | |
|---|---|---|---|---|
| | | 0.6 | 0.525 | 0.50 |
| $\frac{f_{pu} A_{ps}}{f_{cu} bd}$ | 0.05 | 1.00 | 1.00 | 1.00 |
| | 0.0624 | | 1.00 | |
| | 0.1 | 1.00 | 1.00 | 1.00 |

Table 12.4

$\therefore {f_{pb}}/{0.87 f_{pu}} = 1.00$

$f_{pb}$ = 1537N/mm$^2$

| | | ${f_{pe}}/{f_{pu}}$ | | |
|---|---|---|---|---|
| | | 0.6 | 0.525 | 0.50 |
| $\frac{f_{pu} A_{ps}}{f_{cu} bd}$ | 0.05 | 0.12 | 0.12 | 0.12 |
| | 0.0624 | | 0.147 | |
| | 0.1 | 0.23 | 0.23 | 0.25 |

Table 12.4

$\therefore {x}/{d} = 0.147$

$x$ = 0.147(340) = 50 mm

$d_n$ = 0.45$x$ = 0.45(50) = 22.5 mm

$M_u = f_{pb} A_{ps}(d - d_n) = 1537 \times 900 \times (340 - 22.5) = 440$ kN-m

Since $M_u^R \leq M_u$, untensioned reinforcement is not required.

12.3.8 Design shear resistance of beams

*Maximum design shear stress:*

The maximum design shear stress should be limited to $0.8\sqrt{f_{cu}}$ or 7.0 N/mm$^2$, whichever is the smaller.

*Calculation of design shear resistance:*

The design ultimate shear resistance of the concrete alone $V_c$ should be separately considered for sections uncracked in flexure and sections cracked in flexure, as different formulae are required for evaluation as the behaviour in each zone has characteristic features. For sections with inclined tendons, the effects of the inclination of the tendons should be incorporated. If necessary, shear reinforcement should be provided.

*Sections uncracked in flexure:*

The design ultimate shear resistance of a section uncracked in flexure $V_{co}$ corresponds to the occurrence of a maximum design principal tensile stress at the centroidal axis of the section of $f_t = 0.24\sqrt{f_{cu}}$. In the calculation of $V_{co}$, the design value of the prestress at the centroidal axis should be taken as $0.8f_{cp}$, in which $f_{cp}$ is the compressive stress at the centroidal axis due to prestress, taken as positive. Based on these design values, the value of $V_{co}$ is determined as:

$$V_{co} = 0.67b_v h\sqrt{{f_t}^2 + 0.8f_{cp}f_t} \qquad \text{Equation 12.4}$$

Typical values of $V_{co}$ so determined for concrete of grade C30 to C60 and $f_{cp}$ ranging from 2 to 14 N/mm$^2$ are tabulated in Table 12.5 of the code.

*Sections cracked in flexure:*

The design ultimate shear resistance of a section cracked in flexure $V_{cr}$ may be calculated using the following equation:

$$V_{cr} = \left(1 - 0.55\frac{f_{pe}}{f_{pu}}\right)v_c b_v d + M_0\frac{V}{M} \text{ or } 0.1\sqrt{f_{cu}}b_v d,$$

whichever is the greater $\qquad$ Equation 12.6

where $v_c$ is the design concrete shear stress obtained from Table 6.3 of the code with $A_s$ replaced by $(A_{ps} + A_s)$, $M_0$ is the moment necessary to produce zero stress in the concrete at the extreme tension fibre calculated with only 0.8 of the compressive stress due to prestress taken into account, and $M$ is the design moment due to the loads applied at ultimate limit state.

The zones where the bending moment is less than the cracking moment should be designed so that the shear force carried by the beam is less than $V_{co}$; the zone where the bending moment is greater than the cracking moment should be designed to ensure that the shear force carried by the beam is less than $V_{cr}$. When the bending moment is less than the cracking moment, $V_{cr}$ is always greater than $V$. It is therefore either $V_{co}$ or $V_{cr}$ may be critical and the section must be designed for the worst case.

*Shear reinforcement:*

The amount of shear reinforcement to be provided is given as follows:

Case 1: Where $V < V_c$ throughout the beam:

no requirement for structural elements of minor importance;

minimum links for structural elements of importance.

Case 2: Where $0.5V_c \leq V < V_c + V_r$ in which $V_r = v_r b_r d$ (see Clause 6.1.2 of the code):

minimum links to be provided.

Case 3: Where $V_c + V_t \leq V$:

links to be provided such that $A_{sv} \geq s_v(V - V_c)/0.87 f_{yv} d_t$ in which $d_t$ is the depth from the extreme compression fibre either to the longitudinal bars or to the centroid of the tendons, whichever is the greater.

At both corners in the tensile zone, a link should pass round a longitudinal bar, a tendon, or a group of tendons having a diameter not less than the link diameter. A link should extend as close to the tension and compression faces as possible, with due regard to cover. The spacing of links should not exceed $0.75d_t$ or 4 times the web thickness. If $V$ exceeds $1.8V_c$, the maximum spacing should be reduced to $0.5d_t$.

<u>Worked Example 12.2: Check of shear resistance for prestressed concrete beam</u>

A prestressed concrete beam, as shown in the Figure 12.2, is designed to carry a live load of 26 kN/m over a simple span of 24 m. The beam is post-tensioned by six tendons each of which consists of nine 12.9 mm diameter super strands. Each tendon is stressed initially to 70% of the characteristic strength. The ducts are to be grouted after prestressing. Check the ultimate shear resistance of the beam at a section 2 m from an end support and provide shear reinforcement if necessary.

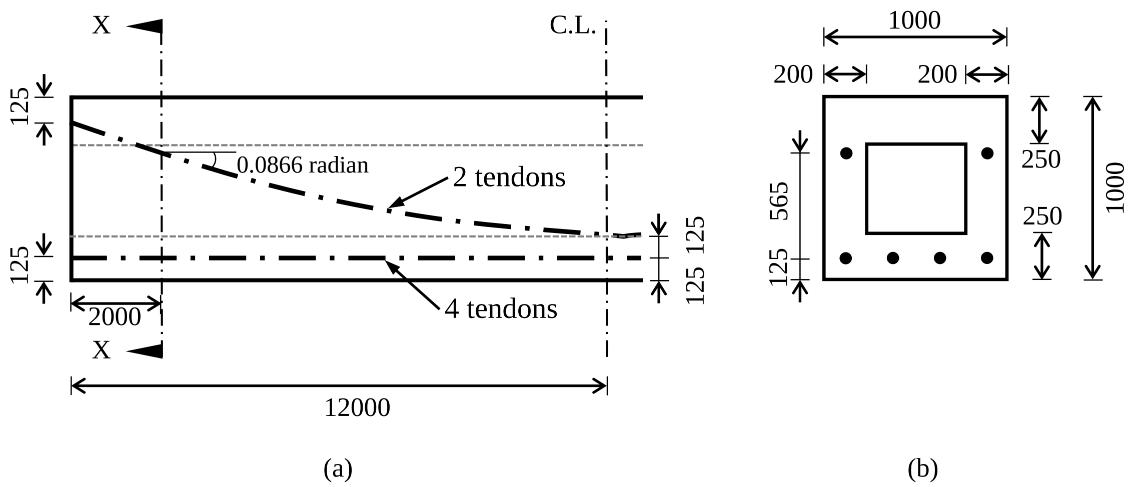



Figure 12.2 (a) Elevation view of the beam (b) Sectional view of X-X

Design data:

| | |
|---|---|
| Concrete characteristic strength: | 50 N/mm$^2$ |
| Characteristic load of 12.9mm diameter super strand | 186 kN |
| Nominal area of 12.9mm diameter super strand | 100 mm$^2$ |
| Ultimate loss of prestress | 25% |
| Duct diameter | 60 mm |

Sectional properties:

Sectional area $A$ = 1000 × 1000 – 600 × 500 = 700,000 mm$^2$

$$I = \frac{bd^3}{12} = \frac{1000(1000)^3}{12} - \frac{600(500)^3}{12} = 7.708 \times 10^{10}\text{mm}^4$$

$$Z = \frac{I}{y} = \frac{7.708\times10^{10}}{500} = 1.542 \times 10^8 \text{ mm}^3$$

$P_e$ = 186 × 9 × 6 × 0.7 × (1–25%) = 5273 kN

$$e = 500 - \frac{4\times125+2\times(125+565)}{6} = 187 \text{ mm}$$

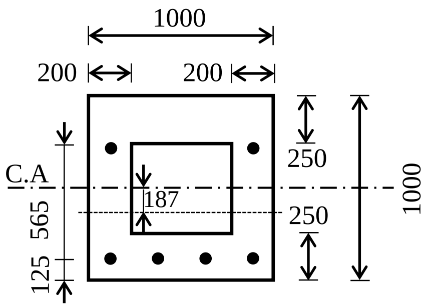



Figure 12.3 Sectional view of X-X with eccentricity e =187mm

Design loadings:

DL = 0.7 × 24 = 16.8 kN/m

LL = 26 kN/m

Ultimate design load = 16.8 × 1.4 + 26 × 1.6 = 65.12 kN/m

At a section 2m from an end support,

Shear force, $V = (65.12 \times 24)\left(\frac{1}{2} - \frac{2}{24}\right) = 651.2$ kN

Bending Moment, $M = \frac{1}{2}\left(\frac{2}{24}\right)\left(1 - \frac{2}{24}\right)(65.12 \times 24)(24) = 1433$ kN-m

$$V_{co} = 0.67 b_v h \sqrt{f_t^{\,2} + 0.8 f_{cp} f_t} \qquad \text{Equation 12.4}$$

where $b_v = \left(200 - {}^{2}/_{3} \times 60\right) \times 2 = 320$ mm

$h$ = 1000 mm

$$f_t = 0.24\sqrt{f_{cu}} = 0.24\sqrt{50} = 1.70 \text{ N/mm}^2$$
$$f_{cp} = {P_e}/{A} = {5273 \times 10^3}/{700{,}000} = 7.533 \text{ N/mm}^2$$
$$V_{co} = 0.67(320)(1000)\sqrt{(1.70)^2 + 0.8(7.533)(1.70)} = 777 \text{ kN}$$

$$V_{cr} = \left(1 - 0.55\frac{f_{pe}}{f_{pu}}\right)v_c b_v d + M_o\frac{V}{M}$$ Equation 12.6

where $f_{pu} = {186 \times 10^3}/{100} = 1860 \text{ N/mm}^2$
$f_{pe} = 1860 \times 0.7 \times (1 - 0.25) = 976.5 \text{ N/mm}^2$
$d$ = 500 + 187 = 687mm

$$M_o = 0.8f_{bp}({I}/{y_b}) = 0.8(13.93)\left({7.708 \times 10^{10}}/{500}\right) = 1718 \text{ kNm}$$

where $f_{bp} = \frac{P_e}{A} + \frac{Pe}{Z_b} = \frac{5273\times10^3}{700000} + \frac{(5273\times10^3)(187)}{1.542\times10^8} = 13.93 \text{ N/mm}^2$

$$v_c = 0.87\left({50}/{25}\right)^{1/3} = 1.096 \text{ N/mm}^2$$

where $\frac{100A_s}{b_v d} = \frac{100(100\times9\times6)}{320\times(500+187)} = 2.47$

| | | $d$ >400 |
|---|---|---|
| $\frac{100A_s}{b_v d}$ | 2.00 | 0.80 |
| | 2.47 | 0.87 |
| | 3.00 | 0.91 |

$$V_{cr} = \left(1 - 0.55\cdot\frac{976.5}{1860}\right)(1.096)(320)(687) + 1718 \times 10^6\left(\frac{651.2\times10^3}{1433\times10^3}\right)$$

$V_{cr}$ = 952 kN

$V$ = 651 kN

$V_{co}$ = 777 kN

$V_{cr}$ = 952 kN

Case 2: $0.5V_c < V \leq V_c + v_r b_v d$

$$\frac{A_{sv}}{s_v} = \frac{v_r b_v}{0.87f_{yv}}.$$ Equation 12.7

where $v_r = 0.4\left({f_{cu}}/{40}\right)^{2/3} = 0.4\left({50}/{40}\right)^{2/3} = 0.464$

$$\frac{A_{sv}}{s_v} = \frac{0.464(320)}{0.87(500)} = 0.341$$

**Y12 (2 legs) at 300mm c/c provided.**

12.3.9 Torsion

Where torsional resistance is necessary for equilibrium or significant torsional stresses may occur, it is necessary to calculate the torsional moment and stresses that could be developed and provide torsional reinforcement accordingly. The method adopted for reinforced concrete beams may generally be used.

## 12.4 Slabs

The recommendations given in Clause 12.3 for beams apply also to slabs, except that shear reinforcement needs not be provided if $V$ is less than $V_c$.

The design of prestressed flat slabs is outside the scope of the code and should be carried out in accordance with appropriate specialist literature.

## 12.5 Columns

For columns in framed structures, where the mean design stress in the concrete section imposed by the tendons is less than 2.0 N/mm$^2$, these may be analysed as ordinary reinforced concrete columns.

## 12.6 Tension members

The tension capacity should be evaluated based on the design strength of the prestressing tendons ($0.87f_{pu}$) and the actual tensile stresses that could be developed in the additional reinforcement (subjected to a maximum value equal to the design strength of the reinforcement, i.e. $0.87f_y$) when the tendons reach their design strength. Strain compatibility should be used to evaluate the tensile stresses in the additional reinforcement. The reason for determining the tensile stresses in the additional reinforcement in this way is that whilst the additional reinforcement should have sufficient ductility to maintain its design strength after yielding until the tendons reach their design strength, the tendons may not have sufficient ductility to maintain their design strength, if they reach their design strength first, until the additional reinforcement also reaches its design strength.

## 12.7 Prestressing

The jacking force should not normally exceed 75% of the characteristic strength of the tendon but may be increased to 80% provided additional consideration is given to safety and to the load-extension characteristics of the tendon. At transfer, the initial prestress should not normally exceed 70% of the characteristic strength of the tendon and in no case should it exceed 75%.

## 12.8 Loss of prestress, other than friction losses

### 12.8.1 General

In the calculation of the design forces in the tendons at various stages of construction, allowance should be made to the possible losses of prestressing forces. Other than friction, losses of prestress may arise from:

- relaxation of the tendon steel;
- elastic deformation and subsequent shrinkage and creep of the concrete;
- draw-in of tendons at anchorage during anchoring; and
- other causes in special circumstances.

### 12.8.2 Relaxation of steel

The long-term loss of prestress due to relaxation of steel may be obtained from the 1000 h relaxation value of the steel by the extrapolation method of multiplying the 1000 h relaxation value with an appropriate relaxation factor. Table 12.6 gives the relaxation factors to be used in the cases of pre-tensioning and post-tensioning. The 1000 h relaxation value should be taken from the manufacturer's appropriate certificate. In the absence of such certificate, the 1000 h relaxation value should be taken as the maximum value stated in the acceptable standards for the product. In special cases, such as with tendons at high temperatures or when subjected to large lateral loads, abnormal relaxation losses may occur. Specialist literature should be consulted in these cases.

### 12.8.3 Elastic deformation of concrete

The immediate loss of prestress due to elastic deformation of the concrete at transfer should be determined using the elastic modulus of the concrete at the time of transfer. In the case of pre-tensioning, the loss of prestress should be calculated on a modular ratio basis using the stress in the adjacent concrete. In the case of post-tensioning if the tendons are not stressed simultaneously, a progressive loss occurs. In such case, the average loss may be calculated on the basis of half the product of the modular ratio and the stress in the concrete adjacent to the tendons averaged along their length or alternatively the loss of prestress of each tendon may be exactly computed on the basis of the sequence of tensioning.

### 12.8.4 Shrinkage of concrete

The loss of prestress due to shrinkage of concrete may be obtained as the product of the shrinkage per unit length of the concrete and the elastic modulus of the tendons. It should be noted that the shrinkage of concrete starts immediately after the curing stops and at the time of tensioning, a significant portion of the total shrinkage might have already taken place.

### 12.8.5 Creep of concrete

The loss of prestress due to creep of concrete may be obtained as the product of the creep per unit length of the concrete adjacent to the tendons and the elastic modulus of the tendons.

### 12.8.6 Draw-in during anchorage

In post-tensioning, allowance should be made for any movement of the tendons at the anchorage when the tensioning force is transferred from the tensioning equipment to the anchorage. Such draw-in loss is particularly important in short members and the allowance made in design should be checked on site.

## 12.9 Loss of prestress due to friction

### 12.9.1 General

In post-tensioning, the tendons will move relative to the surrounding duct during the tensioning operation. The friction arising from such relative movement will cause a reduction in the prestressing force as the distance from the jack increases. In addition, a certain amount of friction will be developed in the jack itself and in the anchorage through which the tendon passes. The variation in force along the length of each tendon should be evaluated so as to determine the tendon forces at the critical sections considered in design. The resulting extension of each tendon should also be calculated taking into account the variation in force along the length of the tendon for later checking on site during the tensioning operation.

### 12.9.2 Friction in jack and anchorage

The friction in jack and anchorage should be determined by calibration of the actual jack and the type of anchorage to be used.

### 12.9.3 Friction in the duct due to unintentional variation from the specified profile

Whether the specified tendon profile is straight or curved, slight unintentional variations from the specified profile cause additional points of contact between the tendon and the sides of the duct and so produce friction. Because of such friction, the prestressing force $P_x$ decreases as the distance from the jacking point $x$ increases. The rate of change of the prestressing force with the distance from the jacking point due to such friction is given by:

$$\frac{\partial P_x}{\partial x} = -KP_x$$

where $K$ is the profile coefficient, the value of which is dependent on the type of duct or sheath employed, the nature and frictional coefficient of its inside surface, the method of forming it and the degree of vibration employed in placing the concrete. The value of $K$ should generally be taken as not less than $33\times10^{-4}$ m$^{-1}$ but where strong rigid sheaths or duct formers are used and closely supported so that they are not displaced during the concreting operation, the value of $K$ may be taken as $17\times10^{-4}$ m$^{-1}$, and for greased strands running in plastic sleeves, the value of $K$ may be taken as $25\times10^{-4}$ m$^{-1}$.

The above equation is given in a different form from that given in the code because the friction due to unintentional variation from the specified profile cannot be considered independently from the friction due to curvature of tendons. The friction due to unintentional variation from the specified profile and the friction due to curvature of tendons have to be considered at the same time because one will affect the other.

### 12.9.4 Friction due to curvature of tendons

The curvature of a tensioned tendon produces radial force acting against the inside surface of the duct or sheath. Such radial force then produces friction along the direction of the tendon and thereby causes the prestressing force $P_x$ to decrease as the distance from the jacking point $x$ increases. The rate of change of the prestressing force with the distance from the jacking point due to such friction is given by:

$$\frac{\partial P_x}{\partial x} = -{}^{\mu P_x}\!/_{r_{ps}}$$

where $\mu$ is the coefficient of friction and $r_{ps}$ is the radius of curvature. The value of $\mu$ is dependent on the type and the surface condition of the tendon and the duct. Typical values of $\mu$ are as follows:

- lightly rusted strand running on unlined concrete duct: 0.55
- lightly rusted strand running on lightly rusted steel duct: 0.30
- lightly rusted strand running on galvanized duct: 0.25
- bright strand running on galvanized duct: 0.20
- greased strand running on plastic sleeve: 0.12

Considering the friction due to unintentional variation from the specified profile and the friction due to curvature of tendons at the same time, i.e. combining the above equations together, the following equation is obtained:

$$\frac{\partial P_x}{\partial x} = -P_x\left(K + \frac{\mu}{r_{ps}}\right)$$

In actual practice, since $r_{ps}$ varies along the length of the tendon, this equation has to be solved by numerical integration, starting at the jacking point where $x = 0$ and $P_x = P_0$. One simple way of evaluating the values of $P_x$ along the length of the tendon is to input the profile of the tendon (in the form of the three-dimensional coordinates of the tendon along its length) into a computer program, which automatically evaluates the curvature at every point along the length, calculates the total frictional loss within each short segment of the tendon and then determines the values of $P_x$ along the length of the tendon by step-by-step integration starting at the jacking point.

12.9.5 Lubricants

If of satisfactory formulation, lubricants may be used to ease the movement of tendons in the ducts. Lower values for $\mu$ may then be used subject to their being determined by trial. The criteria of Clause 10.5.3.1 regarding application of lubricants and subsequent cleansing should then be satisfied if the tendons are subsequently to be bonded into the structure.

## 12.10 Transmission lengths in pre-tensioned members

Clause 8.10.2 should be referred to.

## 12.11 End blocks in post-tensioned members

12.11.1 General

In the design of end blocks, considerations should be given to: (a) bursting forces around individual anchorages; (b) overall equilibrium of the end block; and (c) spalling of the concrete from the loaded face around anchorages. Only the bursting forces are dealt with herein. Special literature should be referred to for consideration of overall equilibrium and spalling of concrete at the loaded face.

12.11.2 Serviceability limit state

At the serviceability limit state, the design bursting force in an individual square end block loaded by a symmetrically placed square bearing plate may be derived from Table 12.7 of the code, which relates the bursting tensile force in each of the two principal directions to the half width of loaded area to half width of end block ratio. This bursting tensile force should be taken as distributed in a region extending from 0.2 times the half width of the end bock to 2 times the half width of the end block from the loaded face, and resisted by reinforcement in the form of spirals or closed links, uniformly distributed throughout this region and acting at a stress of 200 N/mm$^2$. When a large block contains several anchorages, it should be divided into a series of symmetrically loaded prisms and each prism treated in the above manner. However, additional transverse reinforcement should be provided around the group of anchorages to tie the anchorages together and ensure overall equilibrium of the end block.

Special attention should also be paid to end blocks having a cross-section different in shape from that of the general cross-section of the beam. In such case, three-dimensional finite element analysis of the end block may be needed.

### 12.11.3 Ultimate limit state

In the case of members with bonded tendons, no checking of the design bursting tensile force at the ultimate limit state is necessary.

In the case of members with unbonded tendons, the design bursting tensile force at the ultimate limit state should be assessed from Table 12.7 of the code on the basis of the characteristic tendon force; the reinforcement provided to resist this force may be assumed to act at a stress of $0.87f_y$.

## 12.12 Considerations affecting design details

### 12.12.1 General

These considerations are to supplement those given in Section 8.10 of the code.

### 12.12.2 Limitations on area of prestressing tendons

The size and number of prestressing tendons provided should be such that cracking of the concrete would precede ultimate failure of the beam. This is to avoid sudden failure of the beam without any prior signs of failure.

The above requirement may be considered to be satisfied if the ultimate moment of resistance exceeds the moment necessary to produce a flexural tensile stress in the concrete at the extreme tension fibres equal to $0.6\sqrt{f_{cu}}$.

### 12.12.3 Cover to prestressing tendons

*Bonded tendons:*
The nominal cover to be provided should be sufficient to protect the tendons from corrosion and fire, as per the requirements in Sections 4.2 and 4.3 of the code. For pre-tensioned tendons, the ends of individual tendons do not normally require concrete cover and should preferably be cut off flush with the end of the concrete member. For post-tensioned tendons, the equivalent bar size for ducts containing a number of strands should be calculated from the total area of the strands inside the duct and the minimum cover to the outside of the duct should be not less than the minimum dimension of the duct cross-section nor less than half the largest dimension of the duct cross-section.

*Unbonded tendons:*
The cover to the ducts of unbonded tendons should be sufficient to protect the tendons from corrosion and fire, as per the requirements in Sections 4.2 and 4.3 of the code. In any case, the nominal cover to the ducts should not be less than 25 mm.

*External tendons:*
Where external tendons are to be protected by a concrete cover added subsequently, the concrete should be of grade at least C40 and the thickness of the cover should be not less than that required for tendons inside the structural concrete under similar conditions. The concrete cover should be anchored by reinforcement to the prestressed member and should be checked for crack control.

12.12.4 Spacing of prestressing tendons and ducts

There are no additional requirements apart from those given in Clause 8.10.1 of the code.

12.12.5 Longitudinal reinforcement in prestressed concrete beams

Longitudinal reinforcement may be added in prestressed concrete members to increase the strength of sections, to tie up the shear reinforcement or links, or to control cracking.

12.12.6 Links in prestressed concrete beams

Links may be provided in prestressed concrete members to act as shear and torsional reinforcement, to resist the bursting tensile forces in the end blocks of post-tensioned members, or to resisting the bursting forces arising from the anchorage bond stresses in the transmission length of pre-tensioned tendons.

12.12.7 Impact loading

Where a prestressed concrete beam is required to resist impact loading, it should be reinforced with longitudinal reinforcement and closed links, preferably of mild steel. Other methods of design and detailing may be used provided it can be shown that the beam can develop the required ductility.

12.12.13 Ductility

Ductility requirements are added with consideration of beam-column joint, beams and columns. The design basically need to comply with corresponding clauses stated in the code of practice.

# 13 LOAD TESTS OF STRUCTURES OR PARTS OF STRUCTURES

## 13.1 General

Only the testing of whole structures, finished parts of a structure or structural components during the construction phase is covered. Model or prototype testing and appraisal of structure after a certain period of service are not included.

The need for load tests may arise during construction under the following circumstances:

- where the compliance procedures indicate that the materials used may be sub-standard or defective;
- where supervision and inspection procedures indicate poor workmanship on site, producing construction outside the specification and design;
- where there are visible defects, particularly at critical sections or in sensitive structural members;
- where a check is required on the quality of the construction.

It should be recognized that loading a structure to its design ultimate loads may impair its subsequent performance in service, without necessarily giving a true measure of its load carrying capacity. While such overload tests may sometimes be justified, it is generally recommended that the structure should be loaded to a level appropriate to the serviceability limit states. During the tests, sufficient measurements should be taken to calibrate the original design in predicting the ultimate strength and long term performance of the structure.

## 13.2 Test loads

The total load should be not less than 1.0 times the characteristic dead load plus 1.0 times the characteristic imposed load, and should normally be the greater of (a) the sum of the characteristic dead load and 1.25 times the characteristic imposed load or (b) 1.125 times the sum of the characteristic dead and imposed loads.

Test loads should be applied and removed incrementally with a 5 min interval allowed between load increments for recording deformation measurements. At least two cycles of loading and unloading should be applied, with a minimum of 1 h between the two cycles. Consideration may also be given to a third application of load, which is left in position for 24 h.

## 13.3 Assessment of results

The main objective in assessing the results is to compare the measured performance with that expected on the basis of the design calculations. This means that due allowance should be made for any differences in material strength or stress or other characteristics in the as-built structure compared with that assumed in the design.

Steps should be taken to determine these material parameters as accurately as possible using standard control test results and tensioning records etc. Due allowance should also be made for changes in environmental conditions that have occurred during the test.

## 13.4 Test criteria

In assessing test data, the following criteria should be considered:

- The initial deflection and cracking should be in accordance with the design requirements.
- Where significant deflections have occurred under the test loads, the percentage recovery after the second loading cycle should be at least equal to that for the first loading cycle and should be at least 75% for reinforced concrete and Class 3 prestressed concrete structures and 85% for Class 1 and Class 2 prestressed concrete structures.
- The structure should be examined for unexpected defects, which should then be evaluated by recalculation. Comparison between measured and predicted results are important. Where there are significant differences, the first step should be to check that the structure is not carrying the load in a way different from that assumed in the design (due for example to arching action or to the influence of non-structural elements). Material properties may need to be checked as well.

## 13.5 Special tests

Special tests that may be required should be agreed in advance by all the parties concerned.