2 LIMIT STATE DESIGN PHILOSOPHY

2.1 GENERAL

2.1.1 Introduction

Structures should be designed using the methods given in the clauses 2.1.2 to 2.1.6. In applying a particular method, the design of the joints should fulfil the assumptions made in that method without adversely affecting any other part of the structure.

2.1.2 Simple design

The distribution of forces may be determined assuming that members intersecting at a joint are pin connected. Joints should be assumed not to develop moments adversely affecting either the members or the structure as a whole. The necessary flexibility in the connections, other than the bolts, may result in some non-elastic deformation of the materials.

A separate structural system, e.g. bracing, is required to provide lateral restraint in-plane and out-of-plane, to provide sway stability and to resist horizontal forces, see clauses 2.1.3 and 6.3.

2.1.3 Continuous design

Elastic or plastic analysis may be used. In elastic analysis, the joints should have sufficient rotational stiffness to justify analysis based on full continuity. The joints should also be capable of resisting the forces and moments resulting from the analysis.

In plastic analysis, the joints should have sufficient moment capacity to justify analysis assuming plastic hinges in the members. They should also have sufficient rotational stiffness for in-plane stability.

Stability should be properly considered in all the analyses.

For steel with yield strengths greater than 460 N/mm^2 but less than or equal to 690 N/mm^2, the global elastic analysis shall be adopted to design structural members while the cross section and the member resistances are determined in accordance with the cross-section classifications of the members.

2.1.4 Semi-continuous design

Semi-continuous design may be used where the joints have some degree of strength and stiffness which is insufficient to develop full continuity. Elastic or plastic analysis may be used. The moment capacity, rotational stiffness and rotation capacity of the joints shall be based on experimental evidence or advanced elasto-plastic analysis calibrated against tests. This may allow some limited plasticity, provided that the capacity of the bolts or welds is not the failure criterion. On this basis, the design should satisfy the strength, stiffness and in-plane stability requirements of all parts of the structure when partial continuity at the joints is taken into account in determining the moments and forces in the members.

2.1.5 Design justification by tests

In cases where design of a structure or element by calculation in accordance with the methods in clauses 2.1.2 to 2.1.4 is not practicable or is inappropriate the design, in terms of strength, stability, stiffness and deformation capacity, they may be confirmed by appropriate loading tests carried out in accordance with section 16.

2.1.6 Performance-based design

The Code may accept new and alternative methods of design. A performance-based approach to design may be adopted by the Responsible Engineer subject to providing adequate design justification acceptable to the BA that it meets the requirements of the aims of design given in clause 1.2.1. The design may be substantiated by tests if necessary.

2.1.7 Calculation accuracy

In structural design, values of applied loads and properties of materials are not known exactly and this lack of precision is recognized in the partial factor approach. Therefore, in justification calculations or when assessing test results, the observed or calculated result of a test or analysis should be rounded off to the same number of significant figures as in the relevant value recommended in the Code.

2.1.8 Foundation design

The design of foundations should be in accordance with the current Hong Kong Code of Practice on Foundations, recognized geotechnical engineering principles and GEO guidance documents. Foundations should safely accommodate all the forces imposed on them. Attention should be given to the method of connecting the steel superstructure to the foundations and to the anchoring of any holding-down bolts as recommended in section 9.

Forces and moments acting on foundations should be clearly specified and especially as to whether the loads are factored or unfactored and, if factored, the relevant partial load factors for each load in each combination should be stated.

2.2 LIMIT STATE PHILOSOPHY

Limit state design considers the functional limits of strength, stability and serviceability of both single structural elements and the structure as a whole. This contrasts with allowable stress design which considers permissible upper limits of stress in the cross-sections of single members. Limit state design methods may accord more logically with a performance-based design approach.

Limit State Design is based on the requirement that the ‘Resistance’ of the structure (R) should exceed the ‘Load Effects’ (L) for all potential modes of failure, including allowance for uncertainties in load effects and variability in resistance and material properties.

$$\text{i.e.} \quad R > L \qquad (2.1)$$

The load effects, L, shall be determined by normal structural analysis methods for axial, bending, shear or torsion etc. in structural members and components, multiplied by a partial load factor (γ_f) to give an upper bound estimate for load effects. Resistance effects, R, shall be determined by normal strength of materials, geometry of member and material properties. The material yield strength shall be divided by a partial material factor (γ_{m1}) to give a lower bound estimate for material properties. See section 4 for information on partial factors.

Structures should be designed by considering the limit states beyond which they would become unfit for their intended use. Appropriate partial factors should be applied to provide adequate degrees of reliability for ultimate and serviceability limit states. Ultimate limit states concern the safety of the whole or part of the structure whereas serviceability limit states correspond to limits beyond which specified service criteria are no longer met.

The overall level of safety in any design has to cover the variability of material strength, member dimensions and model variability (γ_{m1}), and loading and variations of structural behaviour from that expected (γ_f).

In the Code, the partial material factor has been incorporated in the recommended design strengths for structural steel, bolts and welds.

The values assigned to the load factor (γ_f) depend on the type of load and the load combination. The specified characteristic loads are multiplied by the partial load factor to check the strength and stability of a structure, see section 4.

Examples of limit states relevant to steel structures are given in Table 2.1. In design, the limit states relevant to that structure or part should be considered.

Table 2.1 - Limit states

Ultimate limit states (ULS)	Serviceability limit states (SLS)
Strength (including general yielding, rupture, buckling and forming a mechanism)	Deflection
Stability against overturning, sliding, uplift and sway stability	Human induced vibration
Fire resistance	Wind induced vibration
Brittle fracture and fracture caused by fatigue	Durability

Note:- For cold-formed steel, excessive local deformation is to be assessed under ultimate limit state.

2.3 ULTIMATE LIMIT STATES (ULS)

Ultimate limit states consider the strength and stability of structures and structural members against failure.

For satisfactory design of an element at ultimate limit states, the design resistance or capacity of the element or section must be greater than or equal to the ultimate design load effects. The design resistance is obtained by reducing the characteristic ultimate strength of the material by a partial material factor. The factored design loads are obtained by multiplying the characteristic loads by partial load factors as mentioned in section 4 and the design load effects are obtained by appropriate calculation from the design loads.

2.3.1 Limit state of strength

When checking the strength of a structure, or of any part of it, the specified loads should be multiplied by the relevant partial factors γ_f given in Tables 4.2 and 4.3 of section 4. The factored loads should be applied in the most unfavourable realistic combination for the effect or part under consideration. The principal combinations of loads which should be taken into account are (1) dead load and imposed load, (2) dead load and lateral load effects and (3) dead load, imposed load and lateral load effects.

In each load combination, a γ_f factor of 1.0 should be applied to beneficial dead load which counteracts the effects of other loads, including dead loads restraining sliding, overturning or uplift.

The load carrying capacity of each member and connection, as determined by the relevant provisions of this Code, should be such that the factored loads would not cause failure.

2.3.2 Stability limit states

2.3.2.1 General

Static equilibrium, resistance to horizontal forces and sway stiffness should be checked. The factored loads should be applied in the most unfavourable realistic combination for the part or effect under consideration.

2.3.2.2 Static equilibrium

The factored loads, considered separately or in combination, should not cause the structure, or any part of it (including the foundations), to slide, overturn or lift off its seating. The combination of dead, imposed and lateral loads should be such as to have the most severe effect on the stability limit state under consideration. Account should be taken of variations in dead load which may occur during construction or other temporary conditions.

The design shall also comply with the Building (Construction) Regulations for overall stability against overturning, uplift, and sliding on the basis of working loads.

2.3.2.3 Resistance to horizontal forces

All structures, including portions between expansion joints, should have adequate resistance to horizontal forces in order to provide a practical level of robustness against the effects of incidental loading.

Resistance to horizontal forces should be provided using one or more of the following lateral load resisting systems: triangulated bracing; moment-resisting joints; cantilevered columns; shear walls; properly designed staircase enclosing walls, service and lift cores or similar vertical elements. Reversal of load direction should be considered in the design of these systems.

The cladding, floors and roof should have adequate strength and be properly fixed to the structural framework so as to provide diaphragm action and to transmit all horizontal forces to the lateral load resisting elements (collector points).

Where resistance to horizontal forces is provided by construction other than the steel frame, the steelwork design documents should clearly indicate the need for such construction and state the forces acting on it, see clause 1.2.

2.3.2.4 Sway stiffness and resistance to overall lateral or torsional buckling

All structures should have sufficient sway stiffness so that the vertical loads acting with the lateral displacements of the structure do not induce excessive secondary forces or moments in the members or connections. This requirement shall apply separately to each part of a structure between expansion joints.

Where second order (or "P-Δ") effects are significant, they should be explicitly allowed for in the design of the lateral load resisting parts of the structural system. The system should provide sufficient stiffness to limit sway deformation in any horizontal direction and also to limit twisting of the structure on plan (i.e. to prevent global torsional instability).

When "P-Δ" effects are not significant, the structure may be defined as "non sway" and should still be checked for adequate resistance to notional horizontal forces as defined in clause 2.5.8.

Where moment resisting joints are used to provide sway stiffness, their flexibility should be considered in the analysis of the system. The stiffening effect of masonry infill wall panels or profiled steel sheeting, if present, may be taken into account in the analysis and design. Clause 6.3 describes detailed requirements for the classification of frames.

Member buckling (or "P-δ") should always be considered.

2.3.3 Fatigue

Fatigue need not be considered unless a structure or an element is subjected to numerous fluctuations of stress. Stress changes due to normal fluctuations in wind loading need not be considered. When designing for fatigue, a partial load factor, γ_f of 1.0 should be used.

The principle of fatigue design is given below, further guidance on calculating fatigue resistance and workmanship where fatigue is critical may be found in references given in Annex A1.10. Where fatigue is critical, the workmanship clauses in sections 14 and 15 do not fully cover such cases and all design details should be precisely defined and the required quality of workmanship should be clearly specified.

Situations where fatigue resistance needs to be considered include the following:

- Where there are wind-induced vibrations due to aerodynamic instability. Normal fluctuations in wind loading need not be considered.
- Structural members that support heavy vibratory plant or machinery.
- Members that support cranes as defined in clause 13.7.
- Bridge structures, which will normally be designed to a bridge design code.

Further attention should be paid to the following conditions when considering the fatigue assessment:

- Corrosion/immersion in water will cause a reduction of fatigue life compared to corresponding behaviour in air.
- Very high stress ranges can give rise to low fatigue life.
- The fatigue life will be reduced at regions of stress concentrations; dependent on their geometry welded joints that may have low inherent fatigue life.

- Fatigue life is reduced for some types of thick welded joints compared to thinner joints.
- Fatigue can be caused by repeated fluctuations in thermal stress due to temperature changes. Normal ambient temperature changes will not normally be a problem of fatigue in building structures.

2.3.3.1 Principles of fatigue design

If a component or structure is subjected to repeated stress cycles, it may fail at stresses below the tensile strength and often below the yield strength of the material. The processes leading to this failure are termed 'fatigue'.

Fatigue failure occurs by the slow progressive growth of cracks under the action of repeated fluctuating stresses. The cracks grow incrementally with each cycle of stress range (S). Cracks initiate at the worst stress concentration region and grow in a direction perpendicular to the maximum fluctuation in principal stresses. Final failure occurs when the crack has grown to a size that either the remaining cross section fails by yielding/plastic collapse or the crack has reached a critical size for fracture. Thus the damage done during the fatigue process is cumulative and not recoverable.

Clauses 2.3.3.2 to 2.3.3.4 below describe the basic design philosophy for fatigue tolerant design.

2.3.3.2 S-N Relationships

Most fatigue design rules are presented as a series of S-N curves for particular construction details where N is the available design life expressed as number of cycles of a repeated stress range S. The stress range is the difference between maximum and minimum stress levels. The basic S-N design curves for particular details, e.g. different geometries of welded or bolted connection, are based on experimental laboratory tests on samples of the same geometry and are usually presented to give a specific probability of failure (e.g. mean minus two standard deviations). Figure 2.1 illustrates schematic S-N curves for fatigue cracks growing in a transverse butt weld and from the toe of a transverse fillet weld on a steel plate.

In general, the S-N curve for a particular material and geometry is affected by the mean stress (average of minimum and maximum stresses) and the stress ratio (ratio of minimum to maximum stress). However, in welded joints, the presence of high welding residual stresses means that the mean stress and stress ratio are always high. The basic S-N design curves for welded joints assume that high residual stresses are present and no adjustment should be made for mean stress or stress ratio due to the applied loading.

A classification system links descriptions of the construction detail with the appropriate basic S-N design curve. The classification is dependant upon the joint type, geometry and direction of loading and it relates to a particular location and mode of cracking.

The basic S-N design curves for parent steels are to some extent dependent on the strength of the material, at least as represented by tests on small smooth laboratory specimens. For welded details, however, the fatigue strengths of as-welded components show no increase in available design stress range at the same life for higher strength steels compared to lower strength steels. Fatigue strengths of some types of welded details are reduced by an increase in thickness of the joint and a correction should be applied as follows:

$$S = S_B\left(\frac{t_B}{t}\right)^{1/4} \qquad (2.2)$$

where

S	is the fatigue strength of the joint under consideration, of thickness t,
S_B	is the fatigue strength of the same joint using the basic design curve, derived for thickness t_B, usually taken as 22 mm.
t	is the greater of 16 mm or the actual thickness of the member or bolt;
t_B	is the maximum thickness relevant to the basic S-N design curve.

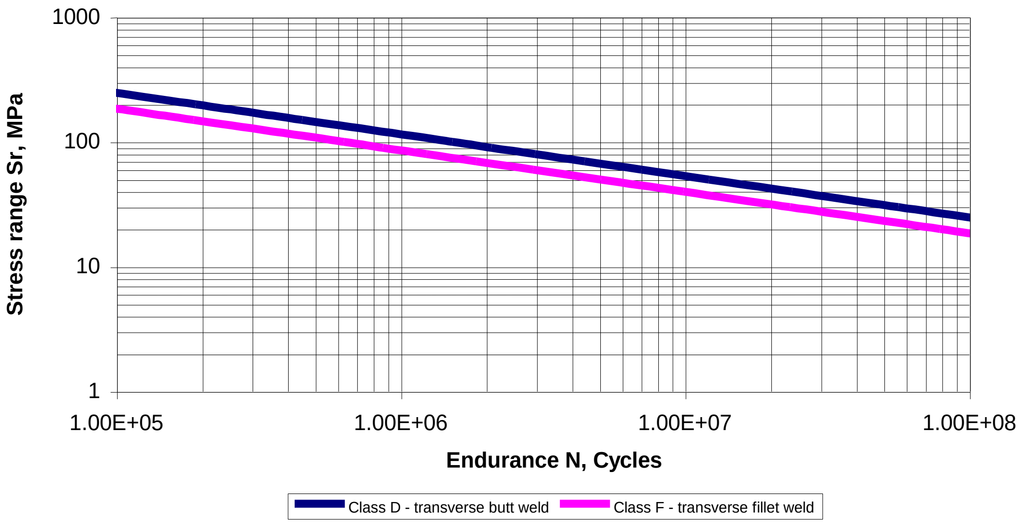

Figure 2.1 - Typical S-N curves for welded steel

The basic S-N design curves are established by constant amplitude test data and are based upon two standard deviations below the means line assuming a log normal distribution, with a normal probability of failure of 2.3%.

2.3.3.3 *Design approaches*

There are two basic approaches to fatigue design:

- Damage tolerant design, and
- Safe life design.

The inherent scatter in fatigue performance and the likelihood of structures being used beyond the intended design life raises the possibility of in-service fatigue cracking. The intent of damage-tolerant (robustness) design is to ensure that should fatigue cracking occur in service, the remaining structure can sustain the maximum working load without failure until the damage is detected.

In situations where regular inspection is not possible, or is otherwise impractical, a safe life design approach is considered appropriate. This is achieved in practice by ensuring that the calculated life is many times greater than the required service life.

2.3.3.4 *Fatigue assessment procedure*

A structural element may contain several fatigue crack initiation sites. The regions of the structure subjected to the highest stress fluctuations and/or containing the severest stress concentrations would normally be checked first. The basic procedure can be summarized as follows:

a) Select the required design working life of the structure, e.g. for a building designed to the Code the design working life is 50 years and for road/bridge this is normally taken as 120 years.

b) Estimate the loading expected in the life of the structure.

c) Estimate the resulting stress history at the detail under consideration.

d) Reduce the *i* th stress history to an equivalent number of cycles n_i of different stress ranges S_{ri} using a cycle counting technique such as the rainflow or reservoir cycle counting methods. See Annex A1.10 for acceptable references.

e) Rank the cycles in descending order of stress range to form a stress spectrum.

f) Classify each detail in order to identify the appropriate S-N curve.

g) Modify the basic S-N design curve as appropriate to allow for variable such as material thickness, corrosion or weld improvement methods.

h) For each stress range of the spectrum S_{ri} determined in (d) and (e) above, determine the available number of cycles N_i from the basic S-N design curve for the detail concerned. Determine the fatigue damage at this stress range as the ratio of the number of cycles applied at this stress range, n_i, to the number of cycles available, N_i. Miner's Law states that the sum of the ratios n_i / N_i for all stress ranges reaches unity for failure. For design purposes, the requirement based on the Miner's summation is therefore as follows:

$$\sum\left(\frac{n_i}{N_i}\right) \leq 1 \qquad (2.3)$$

i) If the Miner's summation is unsatisfactory, modify the peak stress range (and hence all other stress ranges) or the joint classification so as to give a satisfactory value which is equal to or less than 1.0.

2.3.4 Structural integrity and robustness

2.3.4.1 General

To provide structural integrity and robustness and to minimise the risk of localized damage causing progressive collapse, buildings should satisfy the following:

(a) Provide tension continuity tying in both vertically and horizontally.

(b) Ability to resist to a minimum notional horizontal load.

(c) Ability to withstand removal of a vertical element by establishing alternative load paths to limit the extent of damage or collapse. Substantial permanent deformation of members and their connections is acceptable when considering removal of elements.

(d) Design of key elements.

Each part of a building between expansion joints should be treated as a separate building.

2.3.4.2 Principles of tension continuity tying of buildings

The structure of buildings should be effectively tied together at each principal floor level. Every column should be effectively held in position by means of horizontal ties in two directions, approximately at right angles, at each principal floor level supported by that column. Horizontal ties should also be provided at roof level, except where the steelwork only supports lightweight cladding weighing not more than 0.7 kN/m^2 and only carries wind and imposed roof loads.

Continuous lines of ties should be arranged as close as practicable to the edges of the floor or roof and to each column line, see Figure 2.2. At re-entrant corners the tie members nearest to the edge should be anchored into the steel framework as indicated in Figure 2.2. All horizontal ties and their end connections should be robust and ductile.

The horizontal ties may be structural steel members (including those also used for other purposes); or steel bar reinforcement anchored to the steel frame and embedded in concrete; or steel mesh reinforcement in a composite slab with profiled steel sheeting acting compositely with steel beams. The profiled steel sheets should be directly connected to the beams by the shear connectors, see section 10.

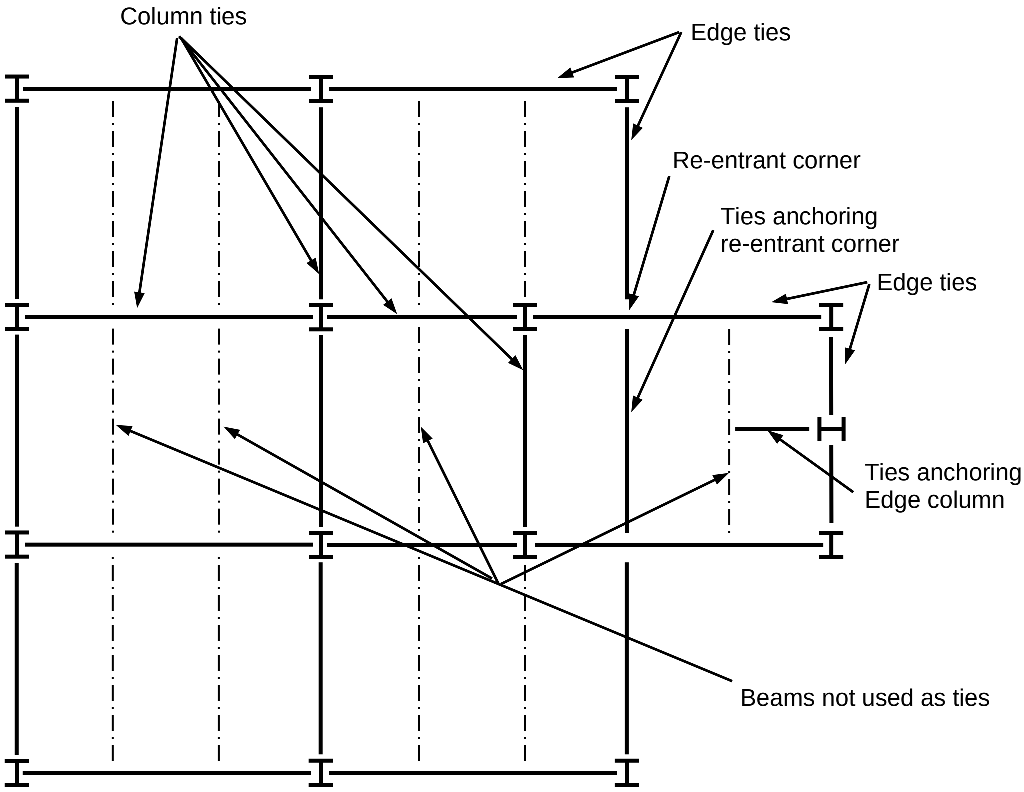

Figure 2.2 - Examples of tying columns of a building

2.3.4.3 *Avoidance of disproportionate collapse*

Steel-framed buildings designed to the Code may be assumed not to be susceptible to disproportionate collapse provided that the following conditions are met.

a) **General tying**. Horizontal ties generally, as described in clause 2.3.4.2, should be arranged in continuous lines wherever practicable, and distributed throughout each floor and roof level in two directions approximately at right angles, see Figure 2.3.

Steel members acting as horizontal ties, and their end connections, should be capable of resisting a factored tensile load not less than 75kN, which need not be considered as additive to other loads. Horizontal ties that consist of steel reinforcement should be designed as recommended in the Code of Practice for Structural Use of Concrete. The tie forces should be calculated as follows:

for internal ties: $0.5\ (1.4G_k + 1.6Q_k)\ s_tL$ but not less than 75 kN; (2.4)

for edge ties: $0.25\ (1.4G_k + 1.6Q_k)\ s_tL$ but not less than 75 kN. (2.5)

where:

G_k is the specified dead load per unit area of the floor or roof;
L is the span of the tie;
Q_k is the specified imposed floor or roof load per unit area;
s_t is the mean transverse spacing of the ties adjacent to that being checked.

This may be assumed to be satisfied if, in the absence of other loading, the member and its end connections are capable of resisting a tensile force equal to its end reaction under factored loads, or the larger end reaction if they are unequal, but not less than 75 kN.

b) **Tying of edge columns**. The horizontal ties anchoring the columns nearest to the edges of a floor or roof should be capable of resisting a factored tensile load, acting perpendicular to the edge, equal to the greater of the tie force as calculated in (a) above, 75 kN or 1% of the maximum factored dead and imposed load in the column immediately above or below that level.

High-rise buildings with outrigger systems or external truss systems often have very large perimeter columns or mega columns. Lateral stability and tying in of such columns requires special consideration as the restraint forces can be large and is referred to clause 13.1.

c) **Continuity of columns**. All columns should be carried through at each beam-to-column connection unless the steel frame is fully continuous in at least one direction. All column splices should be capable of resisting a tensile force equal to the largest factored vertical reaction, from dead and imposed load or from dead, wind and imposed load, applied to the column at a single floor level located between that column splice and the next column splice below.

d) **Resistance to horizontal forces**. Braced bays or other systems for resisting horizontal forces as recommended in clause 2.3.2.3 should be distributed throughout the building such that, in each of the two directions approximately at right angle, no substantial portion of the building is connected at only one point to a system resisting the horizontal forces.

e) **Precast floor units**. Where precast concrete or other heavy floor or roof units are used, they should be properly anchored in the direction of their span, either to each other over a support or directly to their supports as recommended in the Code of Practice for Precast Concrete Construction.

If any of the first three conditions a) to c) is not met, the building should be checked, taking each storey in turn, to ensure that disproportionate collapse would not be initiated by the hypothetical removal, one at a time, of each column. If condition d) is not met, a check should be made in each storey in turn to ensure that disproportionate collapse would not be initiated by the hypothetical removal, one at a time, of each element of the systems providing resistance to horizontal forces. The portion of the building at risk of collapse should not exceed 15% of the floor or roof area or 70 m^2 (whichever is less) at the relevant level and at one immediately adjoining floor or roof level, either above or below it. If the hypothetical removal of a column, or of an element of a system providing resistance to horizontal forces, would risk the collapse of a greater area, that column or element should be designed as a key element, as recommended in clause 2.5.9.

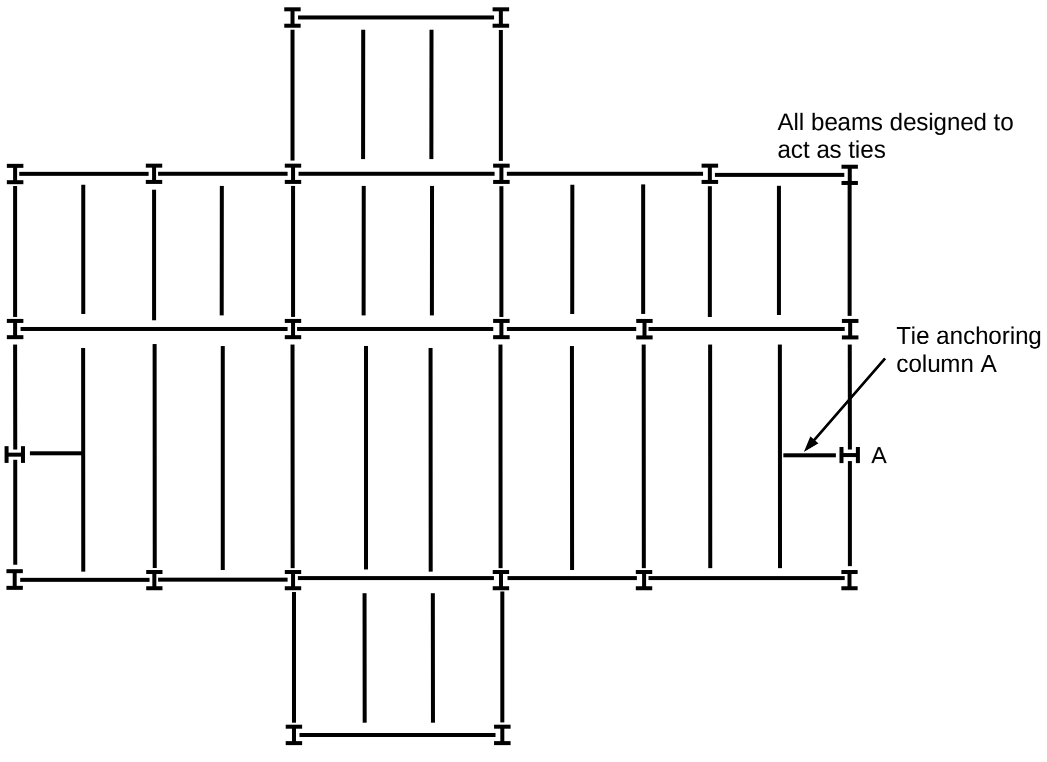

Figure 2.3 - Examples of general tying of a building

2.3.5 Brittle fracture

Brittle fracture can occur in welded steel structures subjected to tensile stresses at low temperatures. In situations where fracture sensitive connection details, inappropriate fabrication conditions or low toughness weld materials, etc, are used, brittle fracture can also occur at normal temperatures. The problem is tackled by specifying steels and welded joints with appropriate grades of fracture toughness, usually implemented in practice by specifying grades of notch ductility in the Charpy test. Higher notch ductility grades are required for thicker steels and joints. Guidance on selection of appropriate grades of notch ductility is given in clause 3.2.

2.4 SERVICEABILITY LIMIT STATES (SLS)

Serviceability limit states consider service requirements for a structure or structural element under normally applied loads. Examples are deflection, human induced vibration, wind induced vibration and durability. They are described in section 5.

For satisfactory design of an element at serviceability limit states, the serviceability design resistance must be greater than or equal to the serviceability design load effects. Typically value of load factors for serviceability calculations is 1.0.

2.4.1 Serviceability loads

Normally the serviceability loads should be taken as the specified characteristic loads, i.e. unfactored.

Exceptional snow load, for cold regions, caused by local drifting on roofs, should not be included in the imposed load when checking serviceability.

In the case of combined wind and imposed loads, only 80% of the full characteristic values need be considered when checking serviceability. In the case of combined horizontal crane loads and wind load, only the greater effect need be considered when checking serviceability.

In calculating most probable deflections for composite elements, it may be necessary to consider creep effects. In such a case, it is necessary to estimate the proportion of live load which is permanent and the proportion which is transitory. For normal domestic and office use, 25% of the imposed load should be considered permanent and 75% considered transitory. For filing and storage, 75% of the imposed load should be considered permanent and for plant floors, the full imposed load should be used.

2.5 LOADING

2.5.1 General

All relevant loads should be considered separately and in such realistic combinations as to give the most critical effects on the elements being designed and the structure as a whole. The magnitude and frequency of fluctuating loads should also be considered.

Loading conditions during erection should be carefully considered. Settlement of supports should be considered where necessary, see clause 2.5.5.

2.5.2 Dead and imposed loading

Refer to the Building (Construction) Regulations for characteristic dead and imposed loads.

For design in countries or regions other than Hong Kong, loads can be determined in accordance with local or national provisions.

2.5.3 Wind loading

Refer to the current Code of Practice on Wind Effects in Hong Kong for characteristic wind loads.

For design in countries or regions other than Hong Kong, wind loads should be determined in accordance with the relevant local or national provisions.

The minimum unfactored wind load should not be less than 1.0% of unfactored dead load in the appropriate load combinations 2 and 3 defined in clause 4.3. This load shall be applied at each floor and calculated from the weight of that floor and associated vertical structure.

For the design of internal structures such as temporary seating in a concert hall, the design unfactored lateral load shall be the greater of 1% of unfactored dead plus imposed loads acting on the floors supporting the internal structures or that obtained from a lateral pressure of 0.5 kN/m^2 multiplied by the appropriate load factor. This pressure should be applied to the enclosing elevation of the structure.

If the specified loads from overhead travelling cranes already include significant horizontal loads, it will not be necessary to include vertical crane loads when calculating the minimum wind load.

2.5.4 Loads from earth and water pressure

Nominal earth and ground-water loads shall be determined in accordance with actual geotechnical conditions and relevant Hong Kong GEO guidance documents.

For design in countries or regions other than Hong Kong, nominal earth and ground water loads shall be determined in accordance with the relevant national or local standards.

2.5.5 Load effects from differential settlement of foundations

In cases where the designer considers the effect of differential settlement of foundations is significant either to an ultimate limit state or to a serviceability limit state, they shall be considered in the design of the structure. The most probable differential settlements may be calculated using appropriate geotechnical methods.

2.5.6 Load effects from temperature change

Where, in the design and erection of a structure, it is necessary to take into account of changes in temperature, it may be assumed that in Hong Kong, the average temperature varies from +0.1ºC to +40.0ºC. The actual range, however, depends on the location, type and purpose of the structure and special consideration may be necessary for structures in other conditions, and in locations outside Hong Kong subjected to different temperature ranges. For some structures such as pre-tensioned rod and cable structural systems, structural stability and designed pre-tension force very much depend on the assumed temperature change and special attention should be paid on design of this structural form, see clause 13.3. Clause 13.3.4.3 provides more detailed guidance for temperatures of elements exposed to sunlight.

2.5.7 Loads from cranes

2.5.7.1 Loads from overhead traveling cranes

The vertical and horizontal dynamic loads and impact effects from overhead travelling cranes should be determined in accordance with clause 13.7 and in considering the limits specified by the crane manufacturer.

Wind loads on outdoor overhead travelling cranes should be obtained from the current HKWC or other appropriate regional wind code. Reference should be made to clause 13.7.1 for cranes under working conditions.

The partial load factors given in Table 13.3 for vertical loads from overhead travelling cranes should be applied to the dynamic vertical wheel loads, i.e. the static vertical wheel loads increased by the appropriate allowance for dynamic effects.

Where a structure or member is subject to loads from two or more cranes, the crane loads should be taken as the maximum vertical and horizontal loads acting simultaneously where this is reasonably possible.

2.5.7.2 *Loads from tower, derrick and mobile cranes*

Where it is required to check the permanent structure for loads imposed from a tower, derrick or mobile crane, the imposed loads should be established from all combinations in consultation with the crane supplier and the building contractor. These combinations shall include loads in service and abnormal loads during adverse wind conditions, e.g. typhoon. Prevention for uplift resistance shall be provided. This data should include loads arising from an envelope of all possible load positions in plan, slew and azimuth angles.

2.5.8 Notional horizontal forces

All practical structures contain imperfections such as lack of verticality and straightness of members. To take into account of this, the lateral load resisting system of all structures should be capable of resisting notional horizontal forces with a minimum of 0.5% of the factored dead and imposed loads applied at the same level as the vertical loads. A minimum notional lateral pressure of 0.5 kN/m^2 shall be used if this gives a higher lateral load than 0.5% of factored dead and imposed load. This pressure should be applied to the enclosing elevation of the structure. No further partial load factor need be applied.

For certain temporary works in construction and sway ultra-sensitive structures, such as internal platform floors, scaffolding, false work and grandstands, a larger minimum horizontal force shall be used. The magnitude of this force shall be the greater of 1.0% of factored dead and live loads applied at the same level or a notional lateral pressure of 1.0kN/m^2 on the enclosing elevation of the structure.

The notional horizontal forces should be assumed to act in any one direction at a time and should be applied at each roof and floor level or their equivalent. They should be taken as acting simultaneously with the factored vertical dead and imposed loads in load combination 1, see section 4.

The notional horizontal forces need not be applied when considering overturning, pattern loads, in combination with other applied horizontal loads or with temperature effects. They need not be taken to contribute to the net reactions at the foundations.

If the specified loads from overhead travelling cranes already include significant horizontal loads, the vertical crane loads need not be included when calculating notional horizontal forces.

Reference should be made to the Code of Practice for Demolition of Buildings for the magnitude of notional horizontal force for supporting structures used in demolition works.

As an alternative to considering notional horizontal forces, the initial imperfections of a structure may be explicitly considered in a non-linear “P-Δ” analysis as described in clause 6.4.

The following table summarises the lateral forces to be considered in design for the principle combinations of load given in clause 4.3.

Table 2.2 - Summary of lateral forces to be considered

Description of load	Principal load combination	Value to be used, larger value of
Notional horizontal force for normal structures	Load combination 1	0.5% of factored dead plus live load or 0.5kN/m^2 notional horizontal pressure. The value used need not be factored further.
Notional horizontal force for temporary works in construction (excluding hoarding structures) and sway ultra-sensitive structures	Load combination 1	1.0% of factored dead plus live load or a minimum notional lateral pressure of 1.0 kN/m^2. The value used need not be factored further.
Lateral loads from wind	Load combinations 2 and 3	Actual wind load, 1.0% of unfactored dead load, or for internal structures a lateral pressure of 0.5kN/m^2. The load used should be multiplied by the appropriate factor for that combination.
Lateral loads from soil and water	Load combinations 2 and 3	Actual values as calculated. The load used should be multiplied by the appropriate factor for that combination.

Note: Refer to the Code of Practice for Demolition of Buildings for appropriate notional horizontal forces for structures under demolition.

2.5.9 Exceptional loads and loads on key elements

Exceptional load cases can arise either from an exceptional load such as an impact from a vehicle (ship, lorry, aeroplane) or explosion, or from consideration of the remaining structure after removal of a key element.

In a building that is required to be designed to avoid disproportionate collapse, a member that is recommended in clause 2.3.4.3 to be designed as a key element should be designed to resist exceptional loading as specified here. Any other steel member or other structural component that provides lateral restraint vital to the stability of a key element should itself also be designed as a key element for the same exceptional loading. The loading should be applied to the member from all horizontal and vertical directions, in one direction at a time, together with the reactions from other building components attached to the member that are subject to the same loading, but limited to the maximum reactions that could reasonably be transmitted, considering the breaking resistances of such components and their connections.

Key elements and connections should be designed to resist an explosion pressure of 34 kN/m^2 or the impact force from a vehicle if considered necessary and possible. Normal nominal design impact forces from vehicles shall be as specified in the current Building (Construction) Regulations.

Table 4.3 contains the load factors and combinations with normal loads to be used in these situations and takes into account of the reduced probability of other loads acting in combination with the exceptional event.

2.5.10 Loads during construction

Loads which arise during construction shall be considered in the design.

2.5.11 Loads on temporary works in construction

The most adverse loading situation arising from the intended construction works should be considered in the design.