13 PERFORMANCE-BASED DESIGN GUIDANCE FOR PARTICULAR TYPES OF STRUCTURES, INCLUDING GUIDANCE ON GENERAL MAINTENANCE OF STEEL STRUCTURES

13.1 HIGH-RISE BUILDINGS

13.1.1 Structural systems for high-rise buildings

This clause mainly focuses on commonly used structural systems in Hong Kong. However, consideration should be given to other structural systems. The principal structural systems for high-rise steel and steel composite buildings in Hong Kong and similar region are:

(a) Steel perimeter columns, floor beams acting compositely with concrete floor and a concrete core providing lateral stability.

(b) Steel perimeter moment frames providing lateral stability, steel and concrete composite floor and concrete core.

(c) Tube in tube systems which are a development of perimeter tube systems.

(d) Outrigger systems comprising a concrete core with a limited number of large perimeter mega columns of composite steel and concrete construction.

(e) External mega trusses or space frames providing the most efficient structural system for super high-rise buildings.

(f) Giant portal frame systems (mega frames) providing lateral stability.

13.1.2 Stability issues for high-rise buildings

13.1.2.1 Overall rigid body stability

Large lateral loads exist on high-rise buildings in a typhoon wind climate. Such buildings should be checked for stability against overall overturning under the Building (Construction) Regulations. Possible uplift or tension between superstructure elements, e.g. columns or core, and pile caps or between pile caps and piles, should be taken into account in the design.

13.1.2.2 Second-order effects

Second-order P-Δ and P-δ effects could be significant for high-rise buildings and should be evaluated and allowed for. They can be considered directly by a second-order analysis using the P-Δ-δ analysis in section 6.

Alternatively, the P-Δ effect should be allowed for by amplifying the moment using equation 6.9 in section 6. The P-δ effect should be considered by the effective length method using the non-sway column buckling length or conservatively assuming the column length as the effective length.

13.1.3 Considerations for particular details

13.1.3.1 Outrigger system

Differential shortening between core and perimeter columns can occur in outrigger or other structural systems. Typically the columns will be highly stressed as compared with the core. Thus the columns will shorten more than the core as gravity loads build up during construction. In addition, concrete elements will continue to creep under load after construction is completed.

Large forces may be induced in the columns and outrigger beams by differential shortening. These should be estimated by carrying out a gravity load analysis and allowed for in the design of the columns and outrigger beams by taken into account the construction sequence. Both elastic and long-term differential shortening caused by creep shall be considered.

Means of adjustment such as a system of jacks and wedges may be provided in order to reduce the magnitude of these forces. The Responsible Engineer shall decide at which stage in the construction process the outrigger shall be locked to the column and then design the permanent structure to safely resist the further forces which arise.

13.1.3.2 Tolerance for lift cores

High-rise buildings may require more stringent specifications on the allowable deviation of lift cores, because of the likelihood of high-speed lifts being used. Non-structural elements such as pipes, claddings, curtain wall, windows etc. should also be checked against sway.

13.1.3.3 Connection ductility in composite frames

Sufficient ductility to allow rotation without brittle failure in extreme events should be provided in composite frame connections.

13.1.3.4 Connection of steel floor beams to concrete cores

Connections between steel floor beams and concrete cores should provide tolerance for erection and sufficient tie capacity for robustness. As a minimum, the tying force should be taken from clause 2.3.4.3 for internal ties, i.e. a value of:-

$$0.5 \times (1.4G_k + 1.6Q_k) \times \text{tie spacing} \times \text{tie span}$$

but not less than 75 kN. The connection detail should be ductile, i.e. allow significant rotation such that the connection can function after the beam sags into a catenary tie. Steel connections from beams to core shall be fire protected even when a fire engineering approach has justified that the floor beams themselves do not require any fire protection.

13.1.4 Considerations for design against extreme events

13.1.4.1 Robustness

A structure should be designed for adequate robustness. Clause 2.3.4 provides guidance on structural integrity, design against progressive collapse and design of key elements.

13.1.4.2 Tying of very large columns

High-rise buildings with outrigger systems or external truss systems often have very large perimeter columns or mega columns. The lateral stability and tying in of such columns requires special consideration as the restraint forces can be large. In accordance with clause 2.3.4.3(c) the restraint force should be 1% of the maximum factored dead and imposed loads in the column. As an alternative an appropriate non-linear buckling analysis may be carried out to evaluate the restraint forces required. This analysis may justify a higher or a lower restraint force than the 1% value.

The vertical tension capacity of such columns should be considered such that the structure as a whole could survive the removal of a section of mega column.

13.1.4.3 Structural considerations for escape routes

Elements such as escape staircases, refuge floors, evacuation lift shafts and the like are required to function for as long as necessary to allow people to leave the building safely should an extreme event occur. This requirement shall be taken into account in the structural design of such elements.

13.1.5 Wind engineering for high-rise buildings

For guidance and requirements on lateral deflections and accelerations of high-rise buildings, refer to clause 5.3.

13.1.5.1 General

Effects of wind on buildings should be considered in structural, foundation and cladding design and should not adversely affect comfort of occupants and pedestrians. Control of deflection and acceleration should follow clause 5.3.4. Vortex shedding and cross-wind response should be considered, especially in the design of slender structures. The damping effect may be considered for evaluation of actual structural response.

13.1.5.2 Wind tunnel test

It is recommended that a wind tunnel test be carried out to study the behaviour of structures of non-conventional forms and in locations where complicated local topography may adversely affect the wind condition. Where particular adjoining or surrounding buildings could significantly influence the local wind profile, the effects of their probable removal should be considered.

13.1.5.3 *Vibration*

Structures should be designed to perform satisfactorily against vibration in ultimate (resonance) and serviceability (annoyance and local damage) limit states. Vibration control of high-rise buildings can be considered under two aspects: deflection and acceleration.

13.1.5.4 *Deflection*

Deflections in a high-rise building should generally satisfy the following requirements, unless their violation can be justified as tolerable.

(1) The deflection under serviceability loads of a high-rise steel and steel-composite building or part should not impair the strength or efficiency of the structure or its components or cause damage to the finishes.

(2) Generally, the serviceability loads should be taken as the unfactored specified values.

(3) When checking for deflections, the most adverse realistic combination and arrangement of serviceability loads should be assumed, and the structure may be assumed to behave elastically.

(4) Suggested limits for the calculated deflections of certain structural members or a building as a whole are given in Table 5.1. Circumstances may arise where greater or lesser values would be more appropriate. In such circumstance, justification to prove its structural suitability is required.

(5) In locations where buildings are close together, the possibility of pounding should be investigated.

13.1.5.5 *Acceleration*

Human response to building motions is a complex phenomenon involving many physiological and psychological factors. Human comfort is generally considered as more measurable by acceleration than other quantities. For high-rise buildings, the highest magnitudes of acceleration generally occur near the top of the building at its first natural frequency, but unacceptable accelerations may occur elsewhere in such buildings and in vibration modes with higher frequencies. Refer also to clause 5.3.4.

13.2 GUIDANCE ON DESIGN OF TRANSMISSION TOWERS, MASTS AND CHIMNEYS

13.2.1 Structural systems for transmission towers, masts and chimneys

The steelwork for the structures of towers, masts and steel chimneys is exposed and needs a high level of corrosion protection due to difficulties of access.

The primary structural systems for masts and towers typically include the following:

(a) Lattice frames

(b) Steel tubes

(c) Cables supporting lattice or tubular masts

Appearance may be highly important in some locations.

Structural weight, buckling stability, method of construction and effects of wind and ice load are key design issues.

Fatigue may need to be considered.

13.2.2 Overall stability of towers, masts and chimneys

Such structures have little redundancy of load path. Failure of single elements is likely to result in collapse. Typically loading from a number of wind directions needs to be considered. The following stability checks should be made:

(a) Overall system overturning.

(b) Overall system buckling.

(c) Local system buckling.

(d) Strength of all members and connections.

(e) System imperfections and lack of fit.

13.2.3 Particular details

Particular details requiring special consideration for such structures are:

(a) Mast bases and anchor blocks.

(b) Cable fixings to mast and to anchor blocks.

(c) Welded connection details with regard to fatigue.

(d) Welded joint detailing to avoid cracking during galvanising or other fabrication and erection processes.

13.2.4 Considerations for design against extreme events

(a) Sabotage, especially in view of the minimal structural redundancy. It is often necessary to fence around access points. Other security protection may be desirable depending on the likely wider consequences of a failure.

(b) Remoteness. Access difficulties can delay inspection and repair of damage.

(c) Hilltop sites increase exposure to wind and other adverse weather events.

(d) In colder regions, ice can build up on lattice elements and cables leading to greatly increased wind and gravity loads.

(e) Aircraft warning painting and/or lighting is likely to be required on many sites.

13.2.5 Serviceability issues

The following serviceability issues shall be addressed for towers and masts:

(a) Wind induced vibrations of antennas, structural elements and cables.

(b) Access for maintenance of steelwork can be very difficult, therefore a high quality protective system should be specified.

(c) Required stiffness for purpose (e.g. microwave alignment).

(d) Access facilities for routine maintenance and inspection shall be designed to take into account of the availability and likely competence of staff trained to climb such structures but should normally include ladders fitted with a fall arrest system and regular platforms to rest and safely place work equipment.

13.2.6 Design issues for steel chimneys

In addition to the guidance given in clauses 13.2.1 to 13.2.5, special attention should be given to the following in the design of steel chimneys and flues:

(a) Wind-excited vibrations should be considered and analyzed by aerodynamic methods. For circular chimneys the simplified method in clause 13.2.8 may be used.

(b) Design should be in accordance with the appropriate provisions of the Code and in the acceptable references in Annex A2.1.

(c) To control buckling in the case of a thin walled chimney with effective height to diameter ratio of less than 21 and diameter to thickness ratio of less than 130, the ultimate compressive stresses in the chimney structure arising from the three principal load combinations shall be limited to a value calculated in accordance with Table 12.2 of clause 12.1.4 which allows for reduced steel strength at elevated temperatures. If this value exceeds 140 N/mm^2, then a value of 140 N/mm^2 shall be used. The value should be reduced further for higher aspect ratios.

(d) Where the temperature is higher than 315°C, a strength reduction factor calculated in accordance with Table 12.2 of clause 12.1.4 should be applied to the design strength in the steel.

(e) Guy cables used as anchorage to a chimney should be positioned at a minimum distance of 3 m below the outlet of the chimney to avoid corrosion from flue gases. These guys should not be considered for strength or stability because of the practical problems of examination and maintenance.

(f) Brackets providing resistance to lateral displacement of the chimney and/or supporting part or all of the weight of the chimney should be positioned at distances not exceeding 6 m apart.

13.2.7 Construction and corrosion protection of steel chimneys

General guidance on corrosion protection is given in clause 5.5. For construction and corrosion protection of chimneys and flues, the relevant parts of acceptable references in Annex A2.1 should be followed.

The exterior and interior surfaces of a steel chimney or flue should have satisfactory protective treatment. Allowance for corrosion should be made in the shell in addition to the thickness obtained from calculations for structural stability. Typically an allowance of 3 mm would be required for chimneys externally fitted with waterproof insulation or cladding and internally lined and protected. For unprotected chimneys, 4.5 mm should be provided for a design life of 10 years and 8 mm for 20 years. Unprotected oil-fired steel chimneys are not recommended.

Bimetallic action may adversely affect a chimney or flue, and should be avoided. If two dissimilar metals have to be connected, a suitable non-conductive and water-impervious material should be placed between them.

13.2.8 Wind-excited vibrations of circular chimneys

Flexible slender structures are subject to vibrations caused by cross wind and along wind action. Structures with a circular cross section, such as chimneys, oscillate more strongly across than along wind.

The following simplified approach may be used for across wind vibration, see also clause 5.3:

(a) The Strouhal critical velocity V_{crit} in metres per second for the chimney is to be determined by:

$$V_{crit} = 5\, D_t\, f \qquad (13.1)$$

where f (in Hz) is the natural frequency of the chimney on its foundations. This may be calculated analytically or from the following approximate formula for the case of a regular cone:

$$f = \frac{500(3D_b - D_t)\left[\dfrac{W_s}{W}\right]^{\frac{1}{2}}}{h^2} \qquad (13.2)$$

and

h is the height of chimney (in m)

D_t is the diameter at top (in m)

D_b is the diameter at bottom (in m)

W is the mass per metre height at top of structural shell including lining or encasing, if any (in kg)

W_s is the mass per meter height at top of structural shell excluding lining (in kg)

(b) If V_{crit} exceeds the design wind velocity in metres per second given by the following formula

$$V = 40.4\,(q)^{0.5} \tag{13.3}$$

where q is the design wind pressure in kN/m^2, severe vibration is unlikely and no further calculation is required.

(c) If V_{crit} is less than the design wind velocity, the tendency to oscillate C may be estimated by the following empirical formula:

$$C = 0.6 + K\left[\frac{10\,D_t^{\,2}}{W} + \frac{1.5\Delta}{D_t}\right] \tag{13.4}$$

where

Δ is the calculated deflection (in m) at the top of the chimney for unit distributed load of 1 kPa.

K is 3.5 for all welded construction, 3.0 for welded with flanged and bolted joints and 2.5 for bolted and riveted or all riveted.

(d) If C is less than 1.0, severe vibration is unlikely. If C is between 1.0 and 1.3 the design wind pressure for the chimney should be increased by a factor C^2. If C is larger than 1.3 stabilizers or dampers should be provided to control the vibrations.

13.3 GLASS AND FAÇADE SUPPORTING STRUCTURES

13.3.1 General

Glass and façade panel supporting structures include curtain wall and structures used in supporting glass wall, skylight, balustrade, canopy etc., which support brittle façade and panels. Special consideration should be given to the design of these supporting structures because of lack of ductility in glass and most façade panel materials. Owing to the limited deflection tolerance of glass and most façade panels, deflection can be both serviceability and ultimate requirements. The buckling strength of some members in trusses not fully restrained along their spans depends sensitively on the effective length which is complex to determine. For design of structures with ultimate strength controlled by buckling, a second-order analysis avoiding the use of effective length assumption should be employed.

13.3.2 Deflection limit

Unless justified by more rigorous calculation to account for the flexibility of supporting members, structural members in support of glass / façade panels should not deflect more than 1/180 of their spans in order to validate the rigid support assumption in glass or cladding panel design.

13.3.3 Requirements

The general requirements for design of glass supporting structures are as follows:

(1) The structures should satisfy general design rules in other sections of the Code.

(2) Elastic design method should be used. Supporting structures should remain elastic under ultimate factored loads.

(3) It should have sufficient deformation capacity to accommodate the movements of main structures and to avoid excessive displacement in panels.

(4) Deflection, strength and stability should be checked under combinations of dead load, imposed load, wind load, temperature and movement of main structure due to loads and creep.

(5) The main supporting members and structures may be suspended to the main structure. If they are supported on main structure along their span or by rigid connections at both ends, their interaction should be considered.

(6) Direct contact between steel or metal structures with brittle or delicate cladding materials such as granite, glass and aluminium panels should be avoided. Separation by flexible materials should be detailed.

(7) Sequential collapse due to failure or breakage of a façade or glass panel should be avoided.

13.3.4 Loadings and actions

13.3.4.1 Load pattern

(1) Critical arrangements of loads should be considered.

(2) In the case of wind load applied on un-braced truss, patch loadings with pressure levels of full and half wind load on a single structure or on different spans/bays of a continuous structure should be considered.

(3) Patterned imposed loads should be considered in the evaluation of maximum induced forces.

13.3.4.2 Loads

The HKWC and Building (Construction) Regulations should be used.

13.3.4.3 Temperature

Temperature load plays a particularly important role in design of glass and façade supporting structures. The temperature range below may be used for local design.

(1) External ambience temperature should be 0-40°C and internal temperature should be 5-35°C.

(2) Surface exposed outside and under sunlight should be considered for a temperature of 0-80°C for dark colour and 0-60°C for light colour.

(3) Surface not exposed outside but under direct sunlight should be considered a temperature range of 10-50°C.

(4) Surface temperature exposed outside should be 0-50°C for clear glass and 0-90°C for tinted glass.

(5) The actual temperature changes of a structure should be determined relatively to the temperature when the structure is installed at site. For example, if the temperature during installation is 20°C, the temperature changes will be +30°C and -10°C in accordance with (3) above.

13.3.4.4 Movements

Movement is an important design consideration for glass supporting structures.

(1) Movements may be one-way like concrete creep, settlement, shrinkage, and may be cyclic due to thermal, moisture, wind loads or imposed loads.

(2) Horizontal movement can be taken as storey height/300, the allowable deflection of the main structure or the calculated deflection. Their largest value should be used.

(3) Vertical movement may be determined from the possible relative deflections between the consecutive floors.

(4) Slot-holes in connections should be designed to prevent loosening by use of locking devices such as locking nuts, locking washers etc.

13.3.5 Tensioning structural systems

In the design of structural systems with tension rods or cables, special attention should be given to their geometrically nonlinear behaviour and their sensitivity to temperature change, support movement and possible creep in the cable itself and in the supporting structures. In addition to conventional consideration of loads, clause 13.3.4 for special load consideration should be referred in design of movement and temperature sensitive structures. Its effects on supporting structures should be considered. Pre-tensioned forces can be applied to the tension rods or cables by means of jacks, turn-buckles for light-duty system or other means. A proper monitoring system should be adopted to ensure applied pre-tension forces are within tolerance.

(1) Only the second-order elastic analysis should be used in force and deflection calculations of pre-tensioned members.

(2) Effects of movement in supporting structures, temperature change and possible creep should be considered for determining the required force under various combined load cases.

(3) In design of tensioning system, load sequence needs careful consideration and slackening in members may not be allowed for cables but permitted in tension rod, provided that its buckling capacity is not exceeded. No tensile stress should exceed the material design strength.

(4) Installation procedure and loading sequence should be considered in the design process.

(5) Combined load cases should include full or 80% pre-tension force whichever the unfavourable.

(6) Three dimensional modelling may be required to investigate into the interaction between tension system and supporting structure of comparable stiffness. Deflections release pre-tension forces in tension rods or cables and they are absorbed by the flexibilities of the supporting beams and the pre-tensioned truss. When concrete structures are used as supports, the long-term and short-term Young's modulus of elasticity, cross-sectional area and second moment of area of concrete members should be adopted appropriately for long-term and short-term loads.

(7) For material such as stainless steel without a distinctive yield point, the 0.2% proof stress should be used.

(8) The thread area of tension rods should be used in the calculation of tensile loads.

(9) Full-scale test to PNAP APP-37 (formerly known as PNAP 106) should be carried out for the critical load cases which can be simulated in laboratory.

The structural behaviour is sensitive to site boundary condition. The test report should be prepared by a qualified engineer with sound background in structural engineering in order to correctly simulate the complex response of the structure in a HOKLAS accredited laboratory or other accredited laboratory which has mutual recognition agreements / arrangements with the HOKLAS or the BA.

13.4 TEMPORARY WORKS IN CONSTRUCTION

Temporary works, particularly scaffolding, are susceptible to collapse, and particular attention should be paid to their design. Causes of collapse of temporary works are often due to buckling of members, instability in structures due to inadequate bracing, excessive eccentricities, differential settlement, or partial failure of foundations. Connections are often made from poor quality welding, with flame-cut rather than sawn ends in bearing and eccentricities.

13.4.1 Design philosophy

For temporary structure whereby the λ_{cr} is less than 5, the second-order analysis as described in section 6 should be carried out irrespective of its height.

Pre-tensioned steel wire and cable can be used when it is properly anchored to prevent slippage. Minimum thickness of members should be 2.0 mm and protection is required against corrosion. Hoarding is not included in this clause.

13.4.2 Second-order effects

The second-order P-Δ effects in Figures 13.1a & 13.1b and P-δ effect in Figure 13.1c are inherent to all practical structures and they are particularly important for temporary slender structures. In the analysis and design, these effects introduce an additional stress to the cross-section of a member and they are required to be accounted for.

In correspondence to the P-Δ and P-δ effects, the system imperfection as out-of-plumbness and member curvatures should be considered. Further, the clearance at joints between scaffolding units should be simulated in an analysis using a slightly deformed structural geometry or by application of an equivalent notional horizontal force.

13.4.3 Out-of-plumbness

The out-of-plumbness inclination ϕ of temporary structure not higher than 10 m should be taken as 1%, i.e.

$$\phi = 0.01 \tag{13.5}$$

For temporary structures higher than 10m, the out-of-plumbness inclination is given by

$$\phi = \frac{0.1}{H} \tag{13.6}$$

in which H is the height of the temporary structure in metres.

Alternatively, an equivalent notional horizontal force equal to the initial inclination times the applied vertical forces at the point of application of the vertical forces can be applied to simulate out-of-plumbness.

The actual structure on site should be checked not to exceed the specified out-of-plumbness in Equation 13.5 or 13.6.

13.4.4 Fitness tolerance

Sleeve and splice tolerance exists for fitting of scaffolding units. A realistic fitness tolerance off the centre line of the original vertical structure should be assumed in an analysis and tolerance should be referred to clause 13.4.8.

13.4.5 Member imperfections

Member imperfection in columns of temporary structures should be taken as,

$$\delta = \frac{L}{500} \tag{13.7}$$

and this value may be reduced when columns are placed in parallel as,

$$\delta = \frac{L}{500} \cdot \frac{1}{\sqrt{n}} \tag{13.8}$$

where n is the number of structural elements arranged parallel to each other and similarly supported and propped, with their deformations of the same magnitude due to systematic influences can be excluded.

13.4.6 Support settlements and flexible supports

Support settlements and flexible supports generate a load re-distribution process and should be avoided. Strong and rigid supports should be provided. If it is not possible, their effects should be estimated and included in an analysis.

13.4.7 Over-turning

Over-turning should be prevented with a minimum factor of safety of 2.0 under working loads.

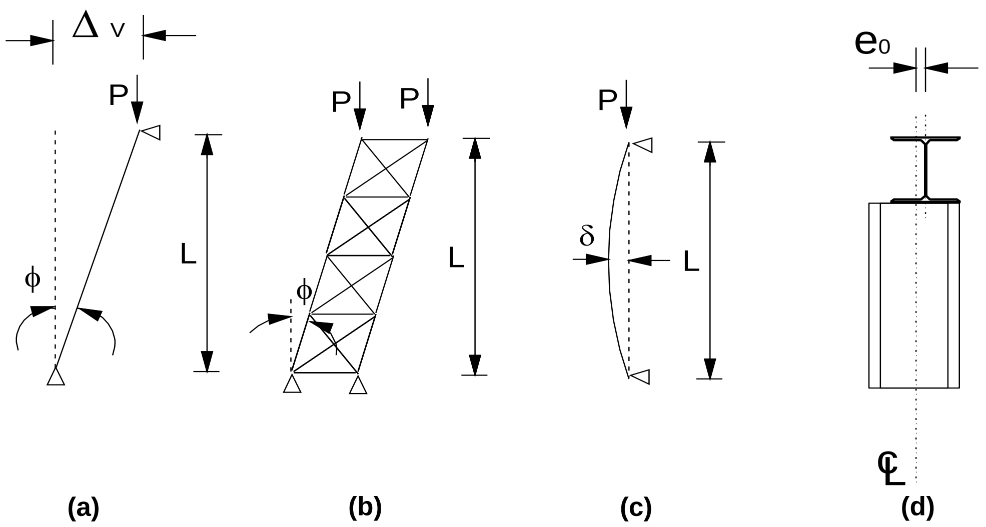

Figure 13.1 - Values of geometric imperfections: (a) & (b) out-of-plumbness, (c) member imperfection and (d) eccentricity

13.4.8 Tolerance and clearance

The following tolerances should be adopted for fabricated temporary structure:

(1) Inclination of a column from vertical (see Figures 13.1a and 13.1.b)

- i) for a column or strut of length L_c < 1450 mm, Δ_v should not exceed 5 mm;
- ii) for a column or strut of length $L_c \geq$ 1450 mm, Δ_v should not exceed $0.0035L_c$ or 25 mm, whichever is the lesser;

  where L_c is the clear height of the column or strut and Δ_v is the inclination from vertical (in mm).

(2) Out-of-straightness of columns and beams (see Figure 13.1c)

- i) for a column or strut of length L < 3350 mm, δ should not exceed 5 mm;
- ii) for a column or strut of length L $\geq$ 3350 mm, δ should not exceed $0.0015L_c$ or 25 mm, whichever is the lesser;

  where L is the clear height of the column or strut and δ is the out-of-straightness of the column or strut (in mm).
- iii) The same tolerance is required for beams, except that δ should not exceed 40mm when span is larger than 3350mm.

(3) The eccentricity of any beam e_o should not exceed 5 mm (see Figure 13.1d).

In circumstances rendering compliance with the above physical tolerances impractical, or unlikely, members should be analysed and designed for such wider tolerance as is considered to be appropriate. Allowable tolerances should be specified on erection/fabrication drawings.

13.4.9 New and used systems

Seriously damaged, cracked, bent or rusted scaffolds should not be used. The condition of the used scaffolds shall be assessed by the Responsible Engineer and only scaffolds of excellent condition can be designed to original design buckling strength.

13.4.10 Module testing

Modular scaffold should be designed and used in accordance with the manufacturer's recommendations. Full justifications including buckling design check by second-order analysis and further tests may be required for critical scaffold modules not covered by the manufacturer's recommendations.

Module testing for the proprietary scaffolding and temporary structural unit should be carried out for height not previously tested or substantiated in manufacturer's manual in order to confirm the accuracy of computed design resistance.

13.5 LONG SPAN STRUCTURES

13.5.1 Systems for long span structures

Long span structures refer to structures with high span-to-depth ratio commonly used in stadia, roofs over exhibition halls, airports, aircraft hangers and similar buildings providing a large column free space. The steelwork for long span building structures is commonly exposed. Structural weight and buckling stability require special considerations in design. The method and sequence of construction will influence the design and should be properly taken into consideration. The stability of partially completed structure shall be ensured during construction.

13.5.2 Overall stability of long span structures

The following stability checks shall be made for long span steel structural elements:

(a) Overall system buckling.
(b) Member buckling.
(c) Snap through instability.
(d) System imperfections and lack of fit.
(e) Stability during construction.

13.5.3 Particular details

Particular details requiring special consideration are:

(a) Steel masts and their bases.
(b) Cable fixings.
(c) Connections of main truss elements.
(d) Connections of secondary to main trusses to provide restraint against buckling.
(e) Dimensional tolerance of interconnected components forming a large span.

13.5.4 Considerations for design against extreme events

Crowd barriers must be designed to resist large crowd loads without collapse. Long span roof trusses should be designed as key elements.

13.5.5 Serviceability issues

The following serviceability issues shall be addressed for long span structures:

(a) Vibration from crowds. Refer to section 5 of the Code.
(b) Wind induced vibrations of roof elements and cables. Fatigue may need to be checked.
(c) Access for maintenance of roof steelwork can be very difficult therefore a high quality protective system should be specified for the steelwork.
(d) Deflection limits for long span trusses under live and wind loads depend on circumstances. A value of span/360 may be used for preliminary design in the absence of other requirements. Significantly smaller deflection limits will be required for applications such as: aircraft hanger doors and stadia opening roofs.

13.6 FOOTBRIDGES

13.6.1 Design philosophy

A footbridge design should satisfactorily accomplish the objectives of constructability, safety and serviceability.

13.6.2 Loads

In general, footbridges are typically designed for pedestrian loads, which are considered static loads. The nominal design load should be obtained from Building (Construction) Regulations and standards issued by the Highways Department. Loads induced by wind, support settlement and temperature change should be accounted for in the design. For local effects, a concentrated load of 10 kN acting on a square of sides of 0.1 m is specified.

13.6.3 Design for strength, deflection and fatigue

13.6.3.1 Strength

Steel structural members, components and connections of a footbridge shall be so proportioned that the basic design requirements for the ultimate limit state given in the Code are satisfied.

13.6.3.2 Deflection

(a) Structural steel members and components of a footbridge shall be so proportioned that the deflections are within the limits agreed between the client, the designer and related authorities as being appropriate to the intended use of the footbridge and the nature of the materials to be supported.

(b) The limiting values for vertical deflections given in Table 13.1 are illustrated by reference to the simply supported beam shown in Figure 13.2, in which:

$$\delta_{max} = \delta_1 + \delta_2 \prod \delta_0 \quad (13.9)$$

where δ_{max} = the sagging in the final state

δ_0 = the pre-camber (hogging) of the beam in the unloaded state.

δ_1 = the variation of the deflection of the beam due to the permanent loads immediately after loading.

δ_2 = the variation of the deflection of the beam due to the pedestrian loads plus any time dependent deformations due to the permanent loads.

$\prod$ represent an operator of deducing for opposite sign and ignoring for same convention sign.

(c) The values given in Table 13.1 are empirical values. They are intended for comparison with the results of calculations and should not be interpreted as performance criteria.

(d) In Table 13.1, L is the span of the beam, and for cantilever beams, the length L to be considered is twice the projecting length of the cantilever.

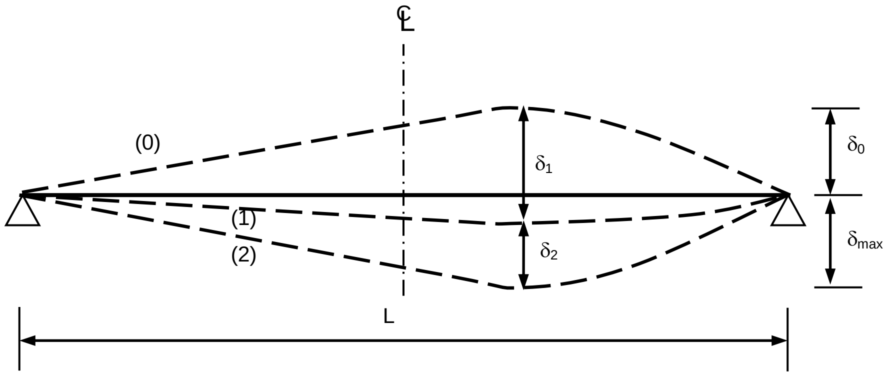

Figure 13.2 - Vertical deflections to be considered

Table 13.1 - Recommended limiting values for vertical deflections

Conditions	Limits (see Figure 13.2)	
	δ_{max}	δ_2
Footbridge Decks generally	L/250	L/300
Footbridge Roofs generally	L/200	L/250
Footbridge Roofs frequently carrying personnel other than for maintenance	L/250	L/300
Where δ_{max} can impair the appearance of the footbridge	L/250	-

13.6.3.3 Fatigue

13.6.3.3.1 General

Footbridges are under fatigue loads from pedestrians. The following measures can be taken to prevent excessive fatigues in footbridges.

(a) For a footbridge, when members are likely subject to crowd-induced vibration, a fatigue check for hot-rolled steelwork, hot finished and cold-finished structural hollow sections shall be required.

(b) Cold-formed steelwork should not be used for footbridges in which fatigue predominates, unless adequate data for the fatigue assessment are available.

(c) Fatigue consideration is included in the Code. Its inclusion does not imply that fatigue is likely to be a design criterion for footbridges.

13.6.3.3.2 Fatigue assessment

No fatigue assessment is required when any of the following conditions is satisfied:

(a) The largest nominal stress range $\Delta\sigma$ satisfies:

$$\gamma_{Ef}\Delta\sigma \leq \frac{26}{\gamma_{Mf}} \text{ N/mm}^2 \tag{13.10}$$

where γ_{Ef} and γ_{Mf} are partial safety factors described below.

(b) The total number of stress cycles N satisfies:

$$N \leq 2\times10^6\left[\frac{36/\gamma_{Mf}}{\gamma_{Ef}\Delta\sigma_{E,2}}\right] \tag{13.11}$$

where γ_{Ef} and γ_{Mf} are partial safety factors described below, and $\Delta_{E,2}$ is the equivalent constant amplitude stress range in N/mm^2. The constant-amplitude stress range that would result in the same fatigue life as for the spectrum of variable-amplitude stress ranges, when the comparison is based on a Miner's summation. For convenience, the equivalent constant amplitude stress range may be related to a total number of 2 million variable amplitude stress range cycles.

(c) For a detail for which a constant amplitude fatigue limit $\Delta\sigma_D$ is specified, the largest stress range (nominal or geometric as appropriate) $\Delta\sigma$ satisfies:

$$\gamma_{Ef}\Delta\sigma \leq \frac{\Delta\sigma_D}{\gamma_{Mf}} \tag{13.12}$$

where γ_{Ef} and γ_{Mf} are partial safety factors described below.

13.6.3.3.3 Partial safety factors

The values of the partial safety factors to be used shall be consistent with the codes used in design. For some special structures, ease of access for inspection or repair, likely frequency of inspection, maintenance and the consequences of failure may require special considerations.

13.6.3.3.4 Partial safety factor for fatigue loading

To take account of uncertainties in the fatigue response analysis, the design stress ranges for the fatigue assessment procedure shall incorporate a partial safety factor γ_{Ef}.

Unless otherwise stated, the following value of γ_{Ef} may be applied to the fatigue loading:

$$\gamma_{Ef} = 1.0 \tag{13.13}$$

13.6.3.3.5 Partial safety factor for fatigue strength

In the fatigue assessment procedure, in order to take account of uncertainties in the fatigue resistance, the design value of the fatigue strength shall be obtained by dividing by a partial safety factor γ_{Mf}.

Recommended values of the partial safety factor γ_{Mf} are given in Table 13.2. These values should be applied to the fatigue strength.

Table 13.2 - Partial safety factor for fatigue strength

Inspection and Access	Primary Components	Secondary Components
Periodic inspection and maintenance. Accessible joint detail.	1.25	1.00
Periodic inspection and maintenance. Poor Accessibility.	1.35	1.15

Notes for Table 13.2:

(i) Inspection may detect fatigue cracks before subsequent damage is caused. Such inspection may be visual unless specified otherwise;

(ii) In-service inspection may not be a requirement of the Code;

(iii) Primary components refer to structural components where local failure of one component leads rapidly to failure of the structure; and

(iv) Secondary components refer to structural components with reduced consequences of failure, such that the local failure of one component does not result in failure of the structure.

13.6.3.3.6 Fatigue assessment

(a) If fatigue assessment is required, fatigue check could be carried out with reference to specialist literature.

(b) A structural health monitoring system may be installed on a footbridge or sensors may be installed on the critical structural components of a footbridge for monitoring.

13.6.4 Vibration

Pedestrians can be adversely affected by the dynamic behaviour of footbridges. In addition to the criteria specified in section 5 on Human-Induced Vibration, the natural frequency of a footbridge shall not be less than 3 Hz. If the natural frequency of a footbridge is less than 3 Hz which may lead to unpleasant vibration, the maximum vertical acceleration, a_v, shall be limited to an appropriate value as given in recognized design guidelines in Annex A2.3 in order to avoid unpleasant vibration.

13.6.5 Bearing design for footbridges

Bearings should be located appropriately to allow sliding movement without causing damage to the structures. For detailed design consideration, local codes such as Structures Design Manual for Highways and Railways should be referred to.

13.7 DESIGN LOADS FROM OVERHEAD RUNWAY CRANES, TOWER AND DERRICK CRANES AND MOBILE CRANES

This section gives guidance on the loads which static cranes apply to building structures. The design of cranes themselves is a specialized activity which is not covered by the Code. Reference may be made to relevant crane design codes given in Appendix A1.10.

13.7.1 Types and classifications of static cranes

13.7.1.1 Overhead runway cranes

Overhead runway cranes comprise a main girder supported on rails at each end. The load is hoisted and carried by a trolley which traverses the main girder. The main girder can traverse along the end rails and thus the load can be moved in two perpendicular directions over the building area.

13.7.1.2 Tower and derrick (or luffing) cranes

Tower cranes consist of a horizontal girder attached to a vertical mast supported from a temporary base attached to the permanent building structure or on a separate foundation at a suitable location, e.g. a lift shaft or lightwell.

Derrick cranes are often used for steel erection in Hong Kong and are typically attached to a mast which is extended with the building in a similar way to that for tower cranes.

13.7.1.3 Mobile cranes

Mobile cranes may be lorry mounted, typically with a telescopic box section boom and supported on jacks attached to the crane by outriggers when in use.

Alternatively cranes may be mounted on a wide tracked base platform, typically with a trussed boom.

13.7.2 Design issues for crane support structures

Cranes will impose large and fluctuating loads on structures. If the crane is heavily used, then fatigue may need to be considered in the design of the supporting structural elements.

The dynamic effect of loads from cranes should be allowed for.

13.7.3 Loading from cranes

13.7.3.1 Loading from overhead travelling cranes

For overhead travelling cranes, the vertical and horizontal dynamic loads and impact effects should be established in consultation with the crane manufacturer.

Loads arise from the dead load of the crane, the live load being lifted, horizontal loads from braking, skewing and buffer collision loads. The loads should be increased by dynamic factors as described below.

In the absence of more precise information an increase of 25% on static vertical loads should be used. A horizontal load of 10% of vertical wheel loads should be taken transverse to the rails and 5% along the rails should be taken.

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. For overhead travelling cranes inside buildings, the following principal combinations of loads should be taken into account in the design of gantry girders and their supports:

Crane combination 1: Dead load, imposed load and vertical crane load;

Crane combination 2: Dead load, imposed load and horizontal crane load;

Crane combination 3: Dead load, imposed load, vertical crane load and horizontal crane load.

Table 13.3 - For normal condition design but with crane load

Load combination (including earth, water and temperature loading where present)	Load Type										
	Dead		Imposed		Earth and water	Wind	Temperature	Vertical crane loads		Horizontal Crane loads	
	G_k		Q_k		S_n	W_k	T_k				
	Adverse	*Beneficial*	*Adverse*	*Beneficial*				*Adverse*	*Beneficial*	*Adverse*	*Beneficial*
1. dead and imposed	1.4	1.0	1.6	0	1.4	-	1.2	1.4	1.0	-	-
	1.4	1.0	1.6	0	1.4	-	1.2	-	-	1.2	0
	1.4	1.0	1.6	0	1.4	-	1.2	1.4	1.0	1.2	0
2. dead and lateral	1.4	1.0	-	-	1.4	1.2	1.2	1.2	1.0	1.2	0
3. dead, lateral and imposed	1.2	1.0	1.2	0	1.2	1.2	1.2	1.2	1.0	1.2	0
4. dead and crane load	1.4	1.0	-	-	1.4	-	1.2	1.6	1.0	-	-
	1.4	1.0	-	-	1.4	-	1.2	-	-	1.6	0
	1.4	1.0	-	-	1.4	-	1.2	1.4	1.0	1.4	0

Where the action of earth or water loads can act beneficially, the partial load factor should not exceed 1.0. (The value of the partial load factor γ_f should be taken such that $\gamma_f \times$ the design earth or water load equals the actual earth or water load)

Where differential settlement is required to be considered a partial load factor of 1.4 shall be used in combinations 1, 2 and 4 and a partial load factor of 1.2 shall be used in combination 3.

13.7.3.2 *Outdoor cranes*

The wind loads on outdoor overhead travelling cranes under working conditions shall be obtained as follows:

(a) Obtain the maximum value of in service wind speed at which the crane is designed to operate.

(b) Calculate the wind load acting on the crane from this and apply to the supporting structure in the most unfavourable direction in combination with other crane loads as specified in clause 13.7.3.1.

The wind loads on outdoor overhead travelling cranes which are not in operation should be calculated using the HKWC.

13.7.3.3 *Load combinations for overhead travelling cranes*

Overhead travelling cranes having vertical and horizontal loads should be considered with other loads in combinations given in Table 13.3. The load factors in Table 13.3 shall be used for loads arising from overhead cranes. The lower value of 1.0 shall be used where the vertical load is beneficial, e.g. against overturning.

13.7.3.4 *Loads from tower and derrick cranes*

The Responsible Engineer will need to consider the temporary loads imposed on the permanent structure from a tower or derrick crane and to check the design of cranes since the cranes may be fabricated from countries with considerable difference in wind load against local condition. The Responsible Engineer shall obtain all the possible loads of the crane from the registered building contractor and tower crane supplier. These combinations shall include loads in service and abnormal loads during typhoon winds. Resistance to uplift shall also be provided for.

13.7.3.5 *Loads from mobile cranes*

Parts of the permanent structure may be required to support mobile cranes during construction and in this case, the Responsible Engineer shall obtain loading data from the contractor or crane supplier. This data should include loads arising from an envelope of all boom positions in plan, slew and azimuth angles.

13.8 GUIDANCE ON MAINTENANCE OF STEEL STRUCTURES

13.8.1 General

Steel structures generally require relatively little maintenance provided that:

(a) Appropriate protection system has been used at the time of construction.

(b) There has been no significant change to the environment envisaged at the time this was specified.

(c) There has been no external event that imposes forces in excess of those allowed for in the design.

The principal reasons for degradation of steel structures are:

(a) Inadequate protection, leading to ongoing corrosion,

(b) Fatigue,

(c) Impact,

(d) Excessive imposed movements, such as differential settlements.

It is the first two of these that generally lead to requirements for maintenance. More significant interventions may be needed, not generally classified as maintenance, for the other events.

By far the most common method of achieving protection to steelwork structures is through the application of a protective coating system. It should be appreciated that the use of coatings is not the only means of preventing steel from corroding.

There are issues that relate to maintenance of buildings in general and are not specific to steelwork structures. Moisture is probably the most common source of problems in buildings, and needs to be controlled. In addition to the provision of adequate ventilation within internal areas, it is essential to keep the roofing and rainwater disposal system (channels, gutters and down pipes) well maintained. Persistent leaks may be detrimental to the structure as well as to the finishes.

13.8.2 Consideration of maintenance in the original design

In specifying protection systems, it is important to ensure that the life of any system, including coatings, meets the client's requirements and the protective system should be properly installed/applied. Consideration should be given to the subsequent work that may be required during the design working life of the building.

Accessibility of steelwork in the completed building, either physically or in practical terms, e.g. where an external stanchion is built into the external wall, needs to be considered when specifying the original protection.

Site conditions may render access for maintenance not possible or disproportionately expensive. In such cases a protective system with a longer life to first maintenance or which assumes that the structure is inaccessible should be considered.

Features where water and dirt may collect should be avoided as these may lead to localised breakdown of protective coatings.

13.8.3 Maintenance of existing structures

For existing construction, there may be an ongoing maintenance regime that is assessed as adequate. Where a building is being taken on initially, the first stage will be to carry out an appraisal of the existing construction and its condition.

Where corrosion has occurred, it is unusual for this to be sufficiently widespread to cause structural damage. However, where structural damage has occurred, the necessary remedial measures fall beyond the scope of routine maintenance.

Once the necessary work has been carried out, an ongoing maintenance schedule will need to take into account of:

(a) The site conditions and restrictions on access;
(b) The use of the building (which may not be the same as for the original construction);
(c) Any particular aesthetic requirements;
(d) The required maintenance interval; and
(e) The scope of maintenance work required, e.g. the need of off-site work for particular elements.

In those structures where fatigue is a design issue, or vibration is anticipated, holding-down bolts should be checked for tightness, any welded joints should be checked for cracking and bearings should be inspected.

13.8.4 Health and safety issues on maintenance

The key health and safety issues are:-

(a) Access
(b) Correct use of materials
(c) Environmental considerations

Access includes consideration of how the necessary maintenance activities will be carried out safely, starting with the initial inspection and then progressing through the various procedures that may be required to ensure that the selected maintenance method can be carried out thoroughly and effectively.

Detailed method statements should be prepared for the different procedures, including risk assessments as relevant.

Manufacturer’s instructions should be followed with respect to storage, handling and use of the different materials. This may include the use of protective clothing and the provision of adequate ventilation while painting and associated preparation is in progress. It is the Responsible Engineer’s duty to ensure that carrying out of the work will not endanger the health and safety of the workers and of the general public. This requires inter alia the provision of screening and the safe disposal of any waste materials.