STRUCTURES DESIGN MANUAL

for Highways and Railways

2013 Edition

Highways Department

The Government of the Hong Kong Special Administrative Region

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FOREWORD

The Structures Design Manual for Highways and Railways provides guidance and sets standards for the design of highway and railway structures in Hong Kong. It has been widely used by practitioners as a reference for local highway and railway structural works since its first publication in August 1993, with second and third editions released in November 1997 and August 2006.

In this 2013 edition, the Manual has been revised for migration from British structural design standards to Eurocodes. Minor amendments taking into account experience gained and feedbacks received on the previous editions are also included.

Eurocodes are the suites of European Standards covering structural design of civil engineering structures, including highway structures. In March 2010, British structural design standards, including BS 5400 which the previous editions of this Manual drew heavily on for requirements, were withdrawn. In anticipation of this, studies were carried out on the adoption of Eurocodes and the associated UK National Annexes for the design of highway structures in Hong Kong. Design approaches, requirements and parameters in Eurocodes considered not suitable for application in Hong Kong have been modified to suit local conditions, material properties and specifications for civil engineering works, and incorporated into this Manual.

This Manual is a guidance document and its recommendations should not be considered as exhaustive. Situation may arise for which considerations and requirements are not fully covered by this Manual. We welcome any comments on this Manual for further improvements. The Bridges and Structures Division of the Highways Department will regularly review and improve on the content of this Manual so that all design standards and guidance will be in line with the most up-to-date practice.

(P. K. LEE)
Chief Highway Engineer / Bridges & Structures
Highways Department

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CHAPTER 1 GENERAL

1.1 SCOPE

(1) This Manual provides guidance and requirements for the design of highway structures and railway bridges in Hong Kong. It shall be used in conjunction with the following Eurocodes and the corresponding UK National Annexes (UK NAs) and Published Documents (PDs) :

BS EN 1990 Eurocode : Basis of Structural Design

BS EN 1991 Eurocode 1 : Actions on Structures

BS EN 1992 Eurocode 2 : Design of Concrete Structures

BS EN 1993 Eurocode 3 : Design of Steel Structures

BS EN 1994 Eurocode 4 : Design of Composite Steel and Concrete Structures

BS EN 1998 Eurocode 8 : Design of Structures for Earthquake Resistance

(2) The relevant Eurocodes, UK NAs and PDs that form the basis of this Manual are listed in Appendix A. Some provisions contained in them are not applicable to Hong Kong conditions, and for these, the recommendations of this Manual, or other appropriate criteria approved for adoption by the Chief Highway Engineer/Bridges and Structures, shall be substituted.

(3) PDs explicitly quoted in this Manual shall be followed. Other PDs not explicitly quoted in this Manual but listed in Appendix A, shall be reviewed and followed where applicable.

(4) Eurocodes shall be used for the design of new and modification of existing highway structures and railway bridges, but not for the structural assessment of existing structures, unless agreed with the Chief Highway Engineer/Bridges and Structures.

(5) In case of conflict, the provisions contained in this Manual shall prevail over those contained in Eurocodes and the corresponding UK NAs and PDs.

(6) The design requirements on concrete, steel and composite concrete and steel highway structures and railway bridges are covered in Chapter 5, Chapter 6 and Chapter 7 of this Manual respectively. For highway structures and railway bridges in other types of materials, design criteria and standards to be adopted shall be agreed with the appropriate authority.

(7) Highway structures designed to this Manual shall be constructed, or executed, in accordance with the General Specification for Civil Engineering Works. Provisions are made such that an acceptable level of reliability will still be maintained without adopting the European Standards and European Technical Approvals for construction products and execution of works in full.

(8) Non-linear analysis generally permitted in Eurocodes shall not be used, unless designers could demonstrate to the satisfaction of the Chief Highway Engineer/Bridges and Structures that either :

   (a) the material properties and the execution of the structures meet the requirements of the European Standards and European Technical Approvals specified in the Eurocodes, or

   (b) appropriate allowance is made in the analysis for any shortfalls in material properties and execution, to ensure sufficient reliability is achieved.

(9) Designers shall be obligated to ensure that provisions given in this Manual are applicable to their designs. For any other design aspects not covered by this Manual, design criteria and standards to be adopted shall be agreed with the Chief Highway Engineer/Bridges and Structures.

1.2 REFERENCE

Unless otherwise stated, references in this Manual to Eurocodes and other similar design standards and manuals shall be to that edition of the document stated in Appendix A. For dated references, subsequent amendments to or revisions of any of these documents shall not apply unless agreed by the Chief Highway Engineer/Bridges and Structures. For undated references, the latest editions of the documents referred to apply (including amendments).

1.3 DEFINITION

1.3.1 Highway Structure

A highway structure is a structure intended to carry highway vehicles, and/or bicycles and pedestrians over, under or through a physical obstruction or hazard, and may be a bridge (which may be in the form of a culvert exceeding 2 m in diameter or span), a flyover, a viaduct, an underpass or a subway. For the purpose of this Manual, walkway covers, cantilever noise barriers, noise enclosures and sign gantries are also considered as highway structures.

1.3.2 Walkway Cover

A walkway cover is an at-grade structure in the form of a roof cover intended to provide shade and shelter from the sun and rain for pedestrians.

1.3.3 Railway Bridge

A railway bridge may be an underbridge or an overbridge. A railway underbridge is a structure intended to carry railway tracks, together with the locomotives and rolling stock using them, over or through a physical obstruction or hazard. A railway overbridge is a structure intended to carry vehicles, pedestrians or services over one or more railway tracks. A railway overbridge may be a highway structure if its primary intention is not for carrying service installations.

1.3.4 Culvert

A culvert is a drainage structure designed as a closed conduit for conveying stormwater from one side of a highway or railway track to the other. A culvert exceeding 2 m in span or diameter corresponds to a small bridge, and shall be treated as a highway structure or railway bridge. A drainage conduit or nullah forming part of a more extensive drainage system which incidentally passes under a highway or railway track at a point or points along its route is a drainage structure, and for the purposes of this Manual is regarded as neither a highway structure nor a railway bridge.

1.3.5 Earth-retaining Structure

A wall designed to hold soil or rock in position is an earth-retaining structure. A wall designed to act as an abutment to a highway structure or railway bridge, or to support an approach to a highway structure or railway bridge, although in itself an earth-retaining structure, shall be treated as part of a highway structure or railway bridge.

1.3.6 Project Office

Project Office is the office in charge of the project or the developer in the case of a private development.

1.3.7 Designer

Designer is the professional, the team of professionals, the company or the organization being responsible for the design.

1.3.8 Checking Engineer

Checking Engineer is the professional, the team of professionals, the company or the organization separate from the Designer and being responsible for the independent check of the design.

1.4 RAILWAY BRIDGES

(1) Before the design of any highway structure crossing a railway track, or of any railway underbridge, is commenced, the requirements of the appropriate railway authority shall be ascertained. Preliminary and detailed drawings, with calculations if required, shall be referred to the appropriate railway authority for comments. The approval of the appropriate railway authority shall be obtained before any work is undertaken.

(2) In the absence of specific comment, the contents of this Manual shall be deemed to apply to railway overbridges and railway underbridges as well as to highway structures.

1.5 APPROVED SUPPLIERS OF MATERIALS AND SPECIALIST CONTRACTORS FOR PUBLIC WORKS

Main contractors engaged on projects involving the supply of special materials or specialist works on highway structures shall either themselves be registered as approved suppliers or specialist contractors in the appropriate category of the List of Approved Suppliers of Materials and Specialist Contractors for Public Works, or shall be required to engage one of the approved suppliers or specialist contractors registered in the category to supply the special materials or to carry out the specialist works on highway structures.

CHAPTER 2 RELIABILITY MANAGEMENT

2.1 RELIABILITY CLASSIFICATION

(1) The provisions given in BS EN 1990 and the UK NA to BS EN 1990 shall be applied together with the modifications given in this Chapter.

(2) All highway structures and railway bridges shall achieve appropriate reliability levels based on the consequence of failure in accordance with Table 2.1 below.

Table 2.1 – Reliability Classification

Consequence Class(1) (BS EN 1990 Table B1)	CC1	CC2	CC3
Reliability Class(2) (BS EN 1990 Table B2)	RC2	RC2	RC3(3)
Design Supervision Level (BS EN 1990 Table B4)	DSL2	DSL2	DSL3
Inspection Level (BS EN 1990 Table B5)	IL3(4)	IL3	IL3

Notes: (1) Unless otherwise stated, CC1, CC2 and CC3 shall be taken as equivalent to the Importance Classes I, II and III for seismic design respectively (see Clause 4.4 for details).

(2) K_{FI} shall be taken as 1.0 and not modified as stated in B3.3 Table B3 of BS EN 1990.

(3) For special bridges (such as suspension bridges, cable stayed bridges, and bridges with span exceeding 150m) and landmark structures in RC3, the required measures to achieve this reliability level shall be determined on a project-specific basis. Possible measures are given in Clause 2.2 of BS EN 1990.

(4) Third party inspection (IL3): Inspection performed by an organisation different from that which has executed the works.

(3) In addition to the design supervision level (DSL) required in Table 2.1, design checking shall be carried out based on the structural category as given in Clause 2.4 and Table 2.2. The more stringent design checking requirement of the two designations must be used.

2.2 DESIGN WORKING LIFE

Unless otherwise stated, the design working life of highway structures and railway bridges shall be 120 years, i.e. design working life category 5 as defined in Clause NA.2.1.1 of the UK NA to BS EN 1990.

2.3 THE USE OF BS EN 1990 ANNEXES

(1) Annex A2 of BS EN 1990 and Clause NA.2.3 of the UK NA to BS EN 1990 shall be followed except as modified in Clause 3.2.

(2) Annexes B, C and D of BS EN 1990 and the corresponding clauses in the UK NA to BS EN 1990 shall not be used, unless otherwise agreed by the Chief Highway Engineer/Bridges and Structures.

2.4 DESIGN CHECKING

2.4.1 General

(1) This Section sets out the guidelines for carrying out independent checking on the design of new highway structures and the associated modification of existing highway structures by consultants or contractors employed by the government. The design checking stipulated below shall also apply to public highway structures which are designed by public organizations (other than the government), private organizations or their agents. These guidelines do not modify the contractual or legal responsibilities of any party for the work carried out, including without limitation the Designer and Checking Engineer as defined in Clause 1.3.

(2) The objective of the independent checking is to ensure :

(a) compliance of the design with the Project Office's requirements, relevant design standards and statutory requirements;

(b) validity of design concepts, methods and assumptions;

(c) applicability, accuracy and validity of the computer programs and models used in the design;

(d) accurate translation of the design into drawings and specifications; and

(e) practicality and adequacy of key details.

2.4.2 Classification of Highway Structures

(1) For design checking purpose, all highway structures shall be classified into Structure Categories I, II and III as shown in Table 2.2. This classification is not rigid and each case shall be decided on its merits having regard to the cost, complexity, safety,

durability and consequences of failure. The Designer shall determine and agree with the Project Office the proposed Category for the highway structures being designed. If necessary, the Project Office or the Designer may approach the Chief Highway Engineer/Bridges and Structures for advice and/or decision on any matters relating to this classification.

Table 2.2 – Classification of Highway Structures for Design Checking

Structure Category	Description
I	**<u>Simple Structures</u>** Structures which contain no departures from or aspects not covered by current standards adopted by Highways Department, and which are either : a) Single simply supported span of less than 20m and having less than 25^{o} skew b) Buried concrete box type structures with less than 8m span c) Retaining walls with a retained height of less than 7m, or d) Noise barrier with a maximum height of 3m
II	**<u>Intermediate Structures</u>** Structures not within the parameters of Structure Categories I and III.
III	**<u>Complex Structures</u>** Structures requiring sophisticated analysis or with any one of the following features : a) High structural redundancy; b) Unconventional design aspects; c) Any span exceeding 80m; d) Skew exceeding 45^{o}; e) Continuous structure with spans exceeding 65m; f) Difficult foundation problems; or g) Difficult construction techniques/ problems.

(2) The Project Office shall arrange with the Designer the checking of a highway structure by a Checking Engineer appropriate to its Category. The Category shall be identified early. As the design evolves, the Designer shall ensure the structure is appropriately classified and seek the agreement of the Project Office to amend its Category and checking arrangements when necessary.

2.4.3 Checking Engineer

(1) The requirements of the Checking Engineer in each category of highway structures are outlined below :

   (a) For Category I structures, an independent check shall be carried out by a qualified professional in the same organization as the Designer who may be from the same design team.

   (b) For Category II structures, an independent check shall be carried out by a qualified professional or checking team in the same organization as the Designer but shall be independent of the design team.

   (c) For Category III structures, an independent check shall be carried out by a checking team from a separate independent organization.

(2) For Categories II and III structures, the Checking Engineer shall be strictly excluded from having direct involvement in the design of the concerned project. In all cases, the Checking Engineer must have sufficient knowledge and experience relating to the type of structures to be checked. The Checking Engineer proposed or appointed by the Designer shall be approved by the Project Office in advance. The Checking Engineer shall exercise reasonable and professional skill, care and diligence at all times in the design checking and that the safety and integrity of the structures shall not be compromised in any way.

(3) Should the Project Office be dissatisfied with the performance of the Checking Engineer at any time, the Project Office may, having given reasonable notice of dissatisfaction, order the dismissal and replacement of the Checking Engineer.

2.4.4 Comment by the Chief Highway Engineer/Bridges and Structures

(1) For Category III structures, the Designer shall at the commencement of the design forward his design approach statement including design concept, design philosophy and outline of mathematical modelling of the structure to the Chief Highway Engineer/Bridges and Structures for comments and make a presentation if required. The Designer shall take account of the Chief Highway Engineer/Bridges and Structures' comments in his design.

(2) The comment by the Chief Highway Engineer/Bridges and Structures will be provided from the viewpoint of design standards and for public interest. It shall not relieve the responsibility of the Designer or the Checking Engineer in any way.

2.4.5 Checking Process

(1) Irrespective of the Category of structures, all design calculations, drawings and specifications shall first be self-checked by the Designer prior to the checking by the Checking Engineer. Also, any computer programs including those developed in-house and spreadsheet applications used in the structural analysis shall be verified and

validated by an appropriate method, and the Designer shall be responsible for such verification and validation.

(2) It is a good practice to start the design checking as early as possible so that the design and checking can proceed together. Also, any disagreements or points of differences can be resolved earlier as the design progresses.

(3) Table 2.3 gives details of the design checking required for each Category of highway structures.

Table 2.3 – Scope of Design Checking

Category	**Scope of Design Checking**
I	a) Check compliance with design codes and standards. b) Carry out arithmetic check on the design calculations. c) Carry out spot checks on critical structural elements. Repetition of numerical calculations is not required if the Checking Engineer can validate the structural adequacy by alternative method or comparison with other similar completed structures. d) Ensure that the design is correctly translated into the drawings and specifications.
II	a) Carry out comprehensive check on drawings with reference to the design calculations. The check will include but not be limited to the design concept, the compliance with design code and standards, the derivation of loadings, method of analysis and design assumptions, the structural adequacy of individual structural elements, stability of the structures and sequence of construction. b) Check/Confirm the applicability, accuracy and validity of all computer programs used by the Designer. c) Check the numerical model, its applicability, input parameters and boundary conditions. d) Carry out separate analytical check on critical structural elements without reference to the design calculations. e) Ensure that the design is correctly translated into the drawings and specifications.
III	a) Derive all loading, design concept, criteria, assumptions and parameters, and sequence of construction from the design document i.e. drawings, design memorandum, specifications, site investigation records, etc. b) Check the compliance with design codes and standards, and limitations if any. c) Check the applicability, accuracy and validity of all computer programs used in design checking. d) Construct computer models, input boundary conditions and parameters and carry out independent structural analysis. e) Prepare an independent set of design check calculations. f) Ensure that the design is correctly translated into the drawings and specifications.

(4) The independent design checking for Category III structures shall be carried out without reference to the design calculations. It is incumbent upon the Checking Engineer to establish the validity of the design assumptions independently. The Checking Engineer would require documents including the design memorandum/manual, drawings, specifications, ground investigation results and other relevant design information for him to carry out the checking. The design memorandum shall contain sufficient information detailing the assumptions made in the design to enable the Checking Engineer to carry out his own independent analysis and assessment and to make direct comparison between his own results and the Designer's design. Major difference in design assumptions should be brought to the attention of the Designer. Although the methods of analysis need not be the same, the Designer and the Checking Engineer should consult with each other to ensure that their calculated results are comparable.

(5) In the event that the design checking reveals errors, omissions or ambiguities in the design, the Checking Engineer shall inform the Designer who shall in turn seek agreement with the Checking Engineer on the course of action required to rectify the design deficiency. The Designer shall make all necessary changes to the design and associated documents, and re-submit them to the Checking Engineer for further review and agreement.

(6) Should the Designer disagree with the Checking Engineer's view, he shall promptly refer the case to the Project Office. Where necessary, advice from an independent expert or the Chief Highway Engineer/Bridges and Structures may be sought.

(7) It must be emphasized that an independent check shall not in any way absolve the Designer from his responsibility and liability for the proper design of highway structures. The independent checking procedures stipulated herein are additional to any in-house design checking by the Designer.

2.4.6 Highway Structures Design and Check Certificate

(1) When the design checking has been completed and all necessary amendments to the design calculations, specifications and drawings have been made and checked by the Checking Engineer, the Designer and the Checking Engineer shall sign the Highway Structures Design and Check Certificate as per the standard form appended in Table 2.4 or as per other form as agreed with the Chief Highway Engineer/Bridges and Structures. Unless there are justifiable reasons acceptable to the Project Office, the Designer shall exercise every effort to ensure that no drawings shall be issued for tendering or construction until the Highway Structures Design and Check Certificate has been accepted by the Project Office.

(2) For Category III structures, a full set of the design submissions and the Highway Structures Design and Check Certificate shall be submitted to the Chief Highway Engineer/Bridges and Structures for audit and record purpose before construction commences. Should the Designer or the Project Office have any difficulties to comply with this requirement under exceptional circumstances, they should seek the special agreement from the Chief Highway Engineer/Bridges and Structures.

(3) For all categories of structures, any amendments to the design deemed necessary which have structural implications following the issue of the Highway Structures Design and Check Certificate shall be checked and certified by an appropriate Checking Engineer. The Designer shall notify the Chief Highway Engineer/Bridges and Structures in case such amendments deviate significantly from the original design intent.

(4) An alternative design by a contractor shall also be subject to design checking if it is to be implemented.

Table 2.4 – Highway Structures Design and Check Certificate

<u>HIGHWAY STRUCTURES DESIGN AND CHECK CERTIFICATE</u>

Agreement No.: ______________________________ **(if appropriate)**
Project Title: ______________________________
Project Office: ______________________________

1. This Design and Check Certificate refers to submission No.
   which comprise
   (a) <u>Highway structures covered by this Certificate</u>
   (nature and description of the submission)

   ____
   ____

   in respect of:
   (description of the highways structures to which the submission refers)

   ____
   ____
   ____
   ____

   (b) Contents of this submission are listed in Schedule A below.

2. Designer's certification

   I / We certify that

   (a) the design of the highway structures, as illustrated and described in the documents listed in Schedule A below, complies with the standards set out in the Agreement or ________________ (any form of agreement as appropriate) and with amendments agreed to by the Director's Representative or ________________ (any relevant authorities as appropriate);

   (b) all reasonable and professional skill, care and diligence have been exercised in designing the highway structures, as illustrated and described in the documents listed in Schedule A below; and

   (c) a self-check has been undertaken and completed to confirm the completeness, adequacy and validity of the design of the highway structures as illustrated and described in documents listed in Schedule A below.

   Signed: ______________________________
   ______________________________ (Name)
   ______________________________ (Position)
   ______________________________ (Organization)
   ______________________________ (Date)

Table 2.4 – Highway Structures Design and Check Certificate (Cont'd)

3. Checking Designer's certification

   (a) I / We certify that the design has been independently checked in accordance with the Structures Design Manual for Highways and Railways using all reasonable skill and care and that I/we am/are satisfied that the design checked complies in all respects with the agreed design criteria.

   (b) I / We further certify that I am/are satisfied that the checking of the above design is completed.

   Signed: ________________________________
   ________________________________ (Name)
   ________________________________ (Position)
   ________________________________ (Organization)
   ________________________________ (Date)

Schedule A

Submission No. _________________ comprises the followings:

Documents: *(Title, reference number and revision)*

____
____
____
____
____
____

Drawings: *(Title, reference number and revision)*

____
____
____
____
____
____
____
____
____

Others: *(Please Specify)*

____
____
____
____
____
____

CHAPTER 3 ACTIONS

3.1 GENERAL

Highway structures and railway bridges shall be designed for the actions and combinations of actions given in Eurocode: Basis of Structural Design (BS EN 1990), Eurocode 1: Actions on Structures – Parts 1 and Part 2 (BS EN 1991-1 and BS EN 1991-2), Eurocode 8: Design of Structures for Earthquake Resistance – Parts 1 and Part 2 (BS EN 1998-1 and BS EN 1998-2) and the corresponding UK National Annexes (UK NAs) and Published Documents (PDs), except where modified by this Manual. A detailed list of the relevant documents is included in Appendix A.

3.2 COMBINATIONS OF ACTIONS

3.2.1 General

(1) All limit states, design situations and combinations of actions given in BS EN 1990 shall be considered. In addition, the following two combinations of actions which are specifically defined for Hong Kong conditions shall also be considered :

   (a) Crack width verification combination as defined in Clause 3.2.2; and

   (b) Tensile stress verification combinations for prestressed concrete members as defined in Clause 3.2.3.

(2) Snow load required in Clause 4.1.1(2) of BS EN 1990 shall be neglected in Hong Kong.

(3) The ψ factors for road bridges given in Table 3.1 shall replace Table NA.A2.1 of the UK NA to BS EN 1990.

(4) The ψ factors for footbridges given in Table 3.2 shall replace Table NA.A2.2 of the UK NA to BS EN 1990.

(5) The design values of actions (EQU) (Set A) given in Table 3.3 shall replace Table NA.A2.4(A) of the UK NA to BS EN 1990.

(6) The design values of actions (STR/GEO) (Set B) given in Table 3.4 shall replace Table NA.A2.4(B) of the UK NA to BS EN 1990.

(7) The design values of actions (STR/GEO) (Set C) given in Table 3.5 shall replace Table NA.A2.4(C) of the UK NA to BS EN 1990.

(8) Frictional forces due to restraint at bearings, $F_{br,k}$, shall be treated as a variable action and considered in combination with other actions defined in BS EN 1990 and BS EN 1991 with the γ and ψ factors given in Table 3.1 to Table 3.5. They need not be considered in combination with any other variable actions except earth pressure where applicable.

(9) Examples of combinations of actions to be considered for typical road bridges are included in Appendix B for reference.

Table 3.1 – Values of ψ Factors for Road Bridges

Action	Group of Loads	Load components	ψ_0	ψ_1	ψ_2
Traffic loads	gr1a[a]	TS	0.75	0.75	0
		UDL	0.75	0.75	0
		Footway and cycle-track loads	0.40	0.40	0
	gr1b[a]	Single axle	0	0.75	0
	gr2	Horizontal forces	0	0	0
	gr3	Pedestrian loads	0	0.40	0
	gr4	Crowd loading	0	_[b]	0
	gr5	Vertical forces from SV and SOV vehicles	0	_[b]	0
	gr6	Horizontal forces from SV and SOV vehicles	0	0	0
Wind forces	F_{Wk}	Persistent design situation	0.50	0.20	0
		During execution	0.80	-	0
	F^*_W	During execution	1.00	-	0
Thermal actions	T_k		0.60	0.60	0.50
Construction loads	Q_c		1.00	-	1.00
Bearing friction	$F_{br,k}$		0	0	0

[a] The recommended values of ψ_0, ψ_1, ψ_2 for gr1a and gr1b are given for roads with traffic corresponding to adjustment factors α_{Qi}, α_{qi}, α_{q1} and β_Q defined in Clause 3.7 of this Manual.

[b] The frequent values of load groups gr4 and gr5 do not need to be considered in accordance with BS EN 1991-2 Clause 4.5.2.

NOTE The ψ_0 factors specified for a group of loads apply to all the component actions in that group, except for gr1a where they are individually specified. The ψ_1 and ψ_2 factors always apply to individual components of loading and the values for a given component are the same in all load groups in which the component occurs.

Table 3.2 – Values of ψ Factors for Footbridges

Actions	Symbol	ψ_0	ψ_1	ψ_2
Traffic loads	gr1	0.40	0.40	0
	Q_{fwk}	0	0	0
	gr2	0	0	0
Wind forces	F_{Wk}	0.30	0.20	0
Thermal actions	T_k	0.60	0.60	0.50
Construction loads	Q_c	1.00	-	1.00
Bearing friction	$F_{br,k}$	0	0	0

Table 3.3 – Design Values of Actions (EQU) (Set A)

Persistent and transient design situation	Permanent actions		Prestress	Leading variable action	Accompanying variable actions	
	Unfavourable	Favourable			Main (if any)	Others
(Eq. 6.10 of BS EN 1990)	$\gamma_{Gj,sup}G_{kj,sup}$	$\gamma_{Gj,inf}G_{kj,inf}$	$\gamma_P P$	$\gamma_{Q,1}Q_{k,1}$		$\gamma_{Q,i}\psi_{0,i}Q_{k,1}$

NOTE 1 For **persistent** design situations the recommended set of values for γ are:

Permanent actions (contributions from the following components should be combined as appropriate)

Concrete self weight	$\gamma_{G,sup} = 1.05$	$\gamma_{G,inf} = 0.95$
Steel self weight	$\gamma_{G,sup} = 1.05$	$\gamma_{G,inf} = 0.95$
Super-imposed dead	$\gamma_{G,sup} = 1.05$	$\gamma_{G,inf} = 0.95$
Road surfacing	$\gamma_{G,sup} = 1.05$	$\gamma_{G,inf} = 0.95$
Ballast	$\gamma_{G,sup} = 1.05$	$\gamma_{G,inf} = 0.95$
Weight of soil	$\gamma_{G,sup} = 1.05$	$\gamma_{G,inf} = 0.95$
Horizontal earth pressure from soil, ground water and free water	$\gamma_{G,sup} = 1.25$	$\gamma_{G,inf} = 0.95$
Self weight of other materials listed in BS EN 1991-1-1:2002, Tables A.1 to A.6	$\gamma_{G,sup} = 1.05$	$\gamma_{G,inf} = 0.95$
Prestressing	γ_P as defined in the relevant design Eurocode or for the individual project	
Variable actions		
Road traffic actions (gr1a, gr1b, gr2, gr5, gr6)	$\gamma_Q = 1.35$	(0 where favourable)
Pedestrian actions (gr3, gr4)	$\gamma_Q = 1.35$	(0 where favourable)
Rail traffic actions	γ_Q to be agreed with appropriate railway authority	
Wind actions		
- Wind velocity pressure derived by simplified procedure (Clause 3.4.2)	$\gamma_Q = 1.55$	(0 where favourable)
- Wind velocity pressure derived by full procedure (Clause 3.4.3)	$\gamma_Q = 2.10$	(0 where favourable)
- Wind actions including aerodynamic effects, where turbulent gust response is considered without vortex excitation	$\gamma_Q = 2.10$	(0 where favourable)
- Wind actions including aerodynamic effects, where vortex excitation is considered (with or without turbulence gust response)	$\gamma_Q = 1.32$	(0 where favourable)
Thermal actions	$\gamma_Q = 1.45$	(0 where favourable)
Bearing friction	$\gamma_Q = 1.30$	(0 where favourable)
Horizontal earth pressure from traffic load surcharge	$\gamma_Q = 1.35$	(0 where favourable)

NOTE 2 For all other actions, not covered in NOTE 1, the partial factors should be determined for the individual project.

NOTE 3 The characteristic values of all unfavourable permanent actions are multiplied by $\gamma_{G,sup}$ and the characteristic values of all favourable permanent actions are multiplied by $\gamma_{G,inf}$ irrespective of whether they arise from a single source, see BS EN 1990 Clause 6.4.3.1(4). See also BS EN 1990 Clause A.2.3.1(2). For design situations involving buried structures, where the stability is highly sensitive to the interaction between the structure and the soil, $\gamma_{G,sup}$ should be applied to unfavourable permanent action effects and $\gamma_{G,inf}$ should be applied to favourable permanent action effects.

NOTE 4 For verification of uplift of bearings of continuous bridges or in cases whether the verification of static equilibrium also involves the resistance of structural elements or the ground γ values may be determined for the individual project as an alternative to separate verifications based on Table 3.3 to Table 3.5 of this Manual, see also BS EN 1990 Clause 6.4.3.1(4).

NOTE 5 For *transient* design situations, during which there is a loss of static equilibrium, $Q_{k,1}$ represents the dominant destabilising variable action and $Q_{k,i}$ represents the relevant accompanying destabilising variable actions.

During execution, if the construction process is adequately controlled, the recommended set of values for γ for the persistent design situations given above may be used with the exceptions set out below :

(A) Where a counterweight is used, the variability of its characteristics may be taken into account, for example, by one or both of the following rules:
- applying a partial factor $\gamma_{G,Inf} = 0.8$ where the self weight is not well defined (e.g. containers);
- by considering a variation of its project-defined location, with a value to be specified proportionately to the dimensions of the bridge, where the magnitude of the counterweight is well defined. For steel bridges during launching, the variation of the counterweight location is often taken equal to ±1 m.

(B) Where loss of equilibrium could result in multiple fatalities (for example bridges constructed over railways or motorways), partial factors for permanent actions affecting stability ($\gamma_{G,sup}$ and $\gamma_{G,inf}$), should be increased to 1.15 and decreased to 0.85 respectively.

Table 3.4 – Design Values of Actions (STR/GEO) (Set B)

Persistent and transient design situation	**Permanent actions**		**Prestress**	**Leading variable action**	**Accompanying variable actions**	
	Unfavourable	Favourable			Main (if any)	Others
(Eq. 6.10 of BS EN 1990)	$\gamma_{Gj,sup}G_{kj,sup}$	$\gamma_{Gj,inf}G_{kj,inf}$	$\gamma_P P$	$\gamma_{Q,1}Q_{k,1}$		$\gamma_{Q,i}\psi_{0,i}Q_{k,1}$

NOTE 1 For **persistent** design situations the recommended set of values for γ are:

Permanent actions (contributions from the following components should be combined as appropriate)

Concrete self weight	$\gamma_{G,sup} = 1.35$	$\gamma_{G,inf} = 0.95$
Steel self weight	$\gamma_{G,sup} = 1.20$	$\gamma_{G,inf} = 0.95$
Super-imposed dead	$\gamma_{G,sup} = 1.50$	$\gamma_{G,inf} = 0.95$
Road surfacing	$\gamma_{G,sup} = 1.20$	$\gamma_{G,inf} = 0.95$
Ballast	$\gamma_{G,sup} = 1.35$	$\gamma_{G,inf} = 0.95$
Weight of soil	$\gamma_{G,sup} = 1.35$	$\gamma_{G,inf} = 0.95$
Horizontal earth pressure from soil, ground water and free water	$\gamma_{G,sup} = 1.65$	$\gamma_{G,inf} = 0.95$
Self weight of other materials listed in BS EN 1991-1-1:2002, Tables A.1 to A.6	$\gamma_{G,sup} = 1.35$	$\gamma_{G,inf} = 0.95$
Settlement (linear structural analysis)	$\gamma_{G,,set,sup} = 1.20$	$\gamma_{G,,set,inf} = 0.00$
Settlement (nonlinear structural analysis)	$\gamma_{G,,set,sup} = 1.35$	$\gamma_{G,,set,inf} = 0.00$
Prestressing	γ_P as defined in the relevant design Eurocode or for the individual project	

Variable actions

Road traffic actions (gr1a, gr1b, gr2, gr5, gr6)	$\gamma_Q = 1.35$	(0 where favourable)
Pedestrian actions (gr3, gr4)	$\gamma_Q = 1.35$	(0 where favourable)
Rail traffic actions	γ_Q to be agreed with appropriate railway authority	
Wind actions		
- Wind velocity pressure derived by simplified procedure (Clause 3.4.2)	$\gamma_Q = 1.55$	(0 where favourable)
- Wind velocity pressure derived by full procedure (Clause 3.4.3)	$\gamma_Q = 2.10$	(0 where favourable)
- Wind actions including aerodynamic effects, where turbulent gust response is considered without vortex excitation	$\gamma_Q = 2.10$	(0 where favourable)
- Wind actions including aerodynamic effects, where vortex excitation is considered (with or without turbulence gust response)	$\gamma_Q = 1.32$	(0 where favourable)
Thermal actions	$\gamma_Q = 1.45$	(0 where favourable)
Bearing friction	$\gamma_Q = 1.30$	(0 where favourable)
Horizontal earth pressure from traffic load surcharge	$\gamma_Q = 1.35$	(0 where favourable)

NOTE 2 For all other actions, not covered in NOTE 1, the partial factors should be determined for the individual project.

NOTE 3 The characteristic values of all permanent actions from one source may be multiplied by $\gamma_{G,sup}$ if the total resulting action effect from this source is unfavourable, and by $\gamma_{G,inf}$ if the total resulting action effect from this source is favourable. However, where a verification is very sensitive to variations in the magnitude of a permanent action from place to place and also involves the resistance of structural elements or the ground, see Table 3.3 Note 4 of this Manual. See also BS EN 1990 Clauses 6.4.3.1(4) and A2.3.1(2).

NOTE 4 For particular verifications, the values of γ_G and γ_Q may be sub-divided into γ_g and γ_q and the model uncertainty factor γ_{Sd}. A value of $\gamma_{Sd} = 1.15$ can be used except where otherwise determined for the individual project.

Table 3.5 – Design Values of Actions (STR/GEO) (Set C)

Persistent and transient design situation	**Permanent actions**		**Prestress**	**Leading variable action**	**Accompanying variable actions**	
	Unfavourable	Favourable			Main (if any)	Others
(Eq. 6.10 of BS EN 1990)	$\gamma_{Gj,sup}G_{kj,sup}$	$\gamma_{Gj,inf}G_{kj,inf}$	$\gamma_P P$	$\gamma_{Q,1}Q_{k,1}$		$\gamma_{Q,i}\psi_{0,i}Q_{k,1}$

NOTE 1 For **persistent** design situations the recommended set of values for γ are:

Permanent actions (contributions from the following components should be combined as appropriate)

Concrete self weight	$\gamma_{G,sup} = 1.00$	$\gamma_{G,inf} = 1.00$
Steel self weight	$\gamma_{G,sup} = 1.00$	$\gamma_{G,inf} = 1.00$
Super-imposed dead	$\gamma_{G,sup} = 1.00$	$\gamma_{G,inf} = 1.00$
Road surfacing	$\gamma_{G,sup} = 1.00$	$\gamma_{G,inf} = 1.00$
Ballast	$\gamma_{G,sup} = 1.00$	$\gamma_{G,inf} = 1.00$
Weight of soil	$\gamma_{G,sup} = 1.00$	$\gamma_{G,inf} = 1.00$
Horizontal earth pressure from soil, ground water and free water	$\gamma_{G,sup} = 1.20$	$\gamma_{G,inf} = 1.00$
Self weight of other materials listed in BS EN 1991-1-1:2002, Tables A.1 to A.6	$\gamma_{G,sup} = 1.00$	$\gamma_{G,inf} = 1.00$
Settlement (linear structural analysis)	$\gamma_{G,,set,sup} = 1.00$	$\gamma_{G,,set,inf} = 0.00$
Settlement (nonlinear structural analysis)	$\gamma_{G,,set,sup} = 1.00$	$\gamma_{G,,set,inf} = 0.00$
Prestressing	γ_P as defined in the relevant design Eurocode of for the individual project	

Variable actions

Road traffic actions (gr1a, gr1b, gr2, gr5, gr6)	$\gamma_Q = 1.15$	(0 where favourable)
Pedestrian actions (gr3, gr4)	$\gamma_Q = 1.15$	(0 where favourable)
Rail traffic actions	γ_Q to be agreed with appropriate railway authority	
Wind actions		
- Wind velocity pressure derived by simplified procedure (Clause 3.4.2)	$\gamma_Q = 1.30$	(0 where favourable)
- Wind velocity pressure derived by full procedure (Clause 3.4.3)	$\gamma_Q = 1.80$	(0 where favourable)
- Wind actions including aerodynamic effects, where turbulent gust response is considered without vortex excitation	$\gamma_Q = 1.80$	(0 where favourable)
- Wind actions including aerodynamic effects, where vortex excitation is considered (with or without turbulence gust response)	$\gamma_Q = 1.10$	(0 where favourable)
Thermal actions	$\gamma_Q = 1.20$	(0 where favourable)
Bearing friction	$\gamma_Q = 1.30$	(0 where favourable)
Horizontal earth pressure from traffic load surcharge	$\gamma_Q = 1.15$	(0 where favourable)

NOTE 2 For all other actions, not covered in NOTE 1, the partial factors should be determined for the individual project.

NOTE 3 The characteristic values of all permanent actions from one source may be multiplied by $\gamma_{G,sup}$ if the total resulting action effect from this source is unfavourable, and by $\gamma_{G,inf}$ if the total resulting action effect from this source is favourable. However, where a verification is very sensitive to variations in the magnitude of a permanent action from place to place and also involves the resistance of structural elements or the ground, see Table 3.3 Note 4 of this Manual. See also BS EN 1990 Clauses 6.4.3.1(4) and A2.3.1(2).

NOTE 4 For particular verifications, the values of γ_Q may be sub-divided into γ_q and the model uncertainty factor γ_{Sd}. A value of γ_{Sd} between 1.05 and 1.15 should be determined for the individual project.

3.2.2 Crack Width Verification Combination

The SLS quasi-permanent combination required in BS EN 1992-1-1, BS EN 1992-2 and the relevant UK NAs for checking crack width shall be replaced by the Crack Width Verification Combination as specified below. The general format of effects of actions shall be :

$E_d = E\{G_{k,j}\ ;\ P;\ Q_{k,1}\}\ ,\ \ j \geq 1$

in which the combination of actions in { } can be expressed as :

$$\sum_{j\geq 1} G_{k,j} \text{"+"} P \text{"+"} Q_{k,1}$$

NOTE 1 For road bridges, $Q_{k,1}$ is the most onerous of the characteristic value of either traffic load groups gr1a or gr5. When gr5 is considered, the axle loads of the SV196 vehicle (refer to Clause 3.7.2.4) shall be multiplied by 0.56. For cantilever slabs and the top flanges on beam-and-slab, voided slab and box beam construction, gr1b shall be considered.

NOTE 2 For pedestrian and cycle bridges, $Q_{k,1}$ is the characteristic value of gr1 as defined in BS EN 1991-2 Table 5.1.

3.2.3 Tensile Stress Verification Combinations for Prestressed Concrete Members

The SLS quasi-permanent and frequent combinations required in BS EN 1992-1-1, BS EN 1992-2 and the relevant UK NAs for checking decompression of prestressed concrete members shall not be followed, and shall be replaced by the Tensile Stress Verification Combinations as specified below.

(1) Tensile Stress Verification Combination for Prestressed Concrete Members – Case 1: No Tensile Stress Permitted.

The general format of effects of actions shall be :

$E_d = E\{G_{k,j}\ ;\ P;\ Q_{k,1}\}\ ,\ \ j \geq 1$

in which the combination of actions in { } can be expressed as :

$$\sum_{j\geq 1} G_{k,j} \text{"+"} P \text{"+"} Q_{k,1}$$

NOTE 1 For road bridges, $Q_{k,1}$ is the most onerous of the characteristic value of either traffic load groups gr1 or gr5. Where gr5 is considered, the axle loads of the SV196 vehicle (refer to Clause 3.7.2.4) shall be multiplied by 0.56. For cantilever slabs and the top flanges on beam-and-slab, voided slab and box beam construction, gr1b shall be considered.

NOTE 2 For pedestrian and cycle bridges, $Q_{k,1}$ is the characteristic value of gr1 as defined in BS EN 1991-2 Table 5.1.

(2) Tensile Stress Verification Combination for Prestressed Concrete Members – Case 2: Tensile Stress Permitted.

The general format of effects of actions shall be :

$E_d = E\ \{G_{k,j}\ ;\ P;\ Q_{k,1};\psi_{0,i}Q_{k,i}\ \}\quad j \geq 1$

in which the combination of actions in { } can be expressed as :

$$\sum_{j\geq 1} G_{k,j} \text{"+"} P \text{"+"} Q_{k,1} \text{"+"} \sum_{i>1} \psi_{0,i} Q_{k,i}$$

NOTE 1 Leading variable action includes wind action, thermal action, centrifugal action and bearing friction. Traffic action needs not be considered as a leading variable action for this case.

NOTE 2 Where traffic loading is an accompanying variable action, gr5 shall be considered with a ψ_0 =1.0.

NOTE 3 No accompanying variable actions are to be combined with centrifugal or bearing friction actions.

3.3 DEAD LOAD AND SUPERIMPOSED DEAD LOAD

(1) Self-weight and superimposed dead loads shall be determined in accordance with BS EN 1991-1-1 and the UK NA to BS EN 1991-1-1 unless otherwise specified in this Manual.

(2) The self-weight of concrete shall be taken as not less than 25 kN/m^3. If the structural concrete of the deck of a structure is to be used as the running surface, the assessment of dead load shall include allowance for a minimum extra thickness of 25 mm of concrete.

(3) If the running surface is to consist of asphalt, the assessment of superimposed dead load shall include allowance for a minimum thickness of 100 mm of asphaltic surfacing material.

(4) A bridge crossing a railway track may be required to carry overhead electrical supply equipment. Reference should be made to the appropriate railway authority for the extra loading to be carried.

(5) The values of dead load and superimposed dead load assumed for preliminary design purposes shall be carefully checked against the final values, when known, and if necessary, the calculations shall be appropriately amended.

3.4 WIND ACTIONS

3.4.1 General

(1) The provisions given in BS EN 1991-1-4, UK NA to BS EN 1991-1-4 and PD 6688-1-4 for wind actions on highway and railway structures shall be followed unless otherwise specified in this Manual.

(2) The provisions for wind velocity and velocity pressure in BS EN 1991-1-4 as supplemented by the UK NA to BS EN 1991-1-4 are based on wind velocities derived from British records and consequently cannot be used, and must be replaced by the provisions in this Manual which are based on Hong Kong conditions.

(3) Waglan Island, which is exposed to southeasterly winds with a long fetch over open sea, is considered as an exposed site to provide the worst wind loading in Hong Kong. Studies show that since Hong Kong is relatively small in climatic terms, the design wind velocity and pressure derived based on the wind data collected from this site are suitable for the design of conventional bridge structures with a degree of conservatism adopted to provide sufficient levels of safety at all sites in Hong Kong. Table 3.6 below indicates the values of wind velocities which have been used to derive the design wind velocity pressure for Hong Kong.

Table 3.6 – Wind Velocities

Location	Return Period	Maximum Hourly Mean Wind Velocity	Maximum Peak Wind Velocity
	Year	m/s	m/s
Waglan Island	50	44	71
	100	48	78
	200	53	85
Hong Kong Observatory	50	41	68
	100	45	75
	200	50	81
NOTE 1 The values for Hong Kong Observatory are for the period before its surrounding area became built-up, and are representative of an exposed urban location.			

The maximum peak wind velocity is related to the peak velocity pressure by the expression:

$$q_p = \frac{1}{2} \rho v_d^2 = 613 \times 10^{-6} v_d^2$$

where q_p = peak velocity pressure (kN/m²)

v_d = maximum peak wind velocity (m/s)

ρ = air density (taken as 1.226 kg/m^3).

By interpolating from the values for Waglan Island in Table 3.6, the above expression gives a value of 3.8 kN/m² for the peak velocity pressure corresponding to the maximum peak wind velocity of 79 m/s for a 120-year return period at an exposed location.

(4) All bridges should be assessed whether a "dynamic response procedure" as stated in BS EN 1991-1-4 Clause 8.2(1) is needed. The design procedure for such assessment should be in accordance with the UK NA to BS EN 1991-1-4 Clause NA.2.49 and PD 6688-1-4 Annex A, with the local parameters given in Clause 3.4.4 of this Manual.

(5) For bridges need not be designed with a "dynamic response procedure", two methods are provided for the calculation of wind actions on structures. The simpler requirements of Clause 3.4.2 may be applied for the majority of highway structures and railway bridges in Hong Kong. Clause 3.4.3 is to be used for structures where an enhanced level of overall structural reliability against failure from wind loading is desired and, shall be applied for all bridges meeting either of the following criteria :

(a) bridges with any span greater than 100m; or

(b) bridges on Strategic Routes and as designated by the Chief Highway Engineer/Bridges and Structures.

(6) A designer experiencing difficulty in deciding on the applicable clause for wind actions or an appropriate degree of exposure for a particular site shall consult the Chief Highway Engineer/Bridges and Structures for advice.

(7) Background information on the derivation of the wind actions given in this Manual is included in Appendix C for reference.

3.4.2 Simplified Procedure for Determining Peak Velocity Pressure

3.4.2.1 Peak Velocity Pressure for Wind Leading Combinations

(1) The peak velocity pressure q_p shall be obtained from Table 3.7 for bridges designed to the simplified procedure, regardless of the height of the structure above ground.

Table 3.7 – Peak Velocity Pressure q_p for Simplified Procedure

Sheltered Location	Exposed Location
q_p (kN/m^2)	q_p (kN/m^2)
2.5	3.8

(2) The values of peak velocity pressure to be used at locations of intermediate exposure are to be interpolated, by the use of engineering judgment, between the extremes given

for sheltered and exposed locations in Table 3.7. To aid designers in choosing suitable values, descriptions and examples of typical locations are given in Table 3.8.

Table 3.8 – Exposure to Wind – Simplified Procedure

Degree of Exposure	Description	Peak Velocity Pressure q_p (kN/m^2)	Example
1	Sheltered by surrounding buildings and /or topography	2.5	Kowloon Park Drive Flyover
2	Normal exposure	2.8	Castle Road Flyover
3	Elevated situation; not sheltered by buildings or topography	3.3	Tai Po Road Interchange
4	Exposed to north-easterly or south easterly winds across open sea	3.8	Ap Lei Chau Bridge

3.4.2.2 Peak Velocity Pressure for Traffic Leading Combinations

(1) For road bridges, the probability of much traffic being present on a bridge at peak wind velocity exceeding 44 m/s is low and the corresponding peak velocity pressure of 1.2 kN/m^2 may be used in traffic leading combinations. Therefore, as discussed in BS EN 1991-1-4 Clause 8.1(4), the combination value $\psi_0 F_{wk}$ of the wind action on the bridge and on the vehicles travelling on the bridge, should be limited to a value F_W^* determined by taking q_p as 1.2 kN/m^2. Provision given in the UK NA to BS EN 1991-1-4 Clause NA.2.47 shall not be followed.

(2) For railway underbridges, the value of q_p for determining F_W^{**}, which is discussed in BS EN 1991-1-4 Clause 8.1(5), shall be agreed with the appropriate railway authority taking into account the possibility of the presence of railway traffic on the bridge at high wind velocity.

3.4.3 Full Procedure for Determining Peak Velocity Pressure

For bridges to be designed under the full procedure, due account shall be taken of the loaded length under consideration and the height of the structure above ground. Provisions given in this Section shall replace Clause NA.2.56 of the UK NA to BS EN 1991-1-4.

3.4.3.1 Peak Velocity Pressure for Wind Leading Combinations

The peak velocity pressure $q_p(z)$ shall be determined in accordance with Table 3.9.

3.4.3.2 Velocity Pressure on Relieving Areas for Wind Leading Combinations

Where wind on any part of a bridge or element gives relief to the member under consideration, the effective coexistent value of velocity pressure on the parts affording relief shall be determined from Table 3.9 as the appropriate hourly mean velocity pressure q'(z).

3.4.3.3 Peak Velocity Pressure for Traffic Leading Combinations

The peak velocity pressure $q_p(z)$ on those parts of the bridge or its elements on which the application of wind actions increases the effect being considered shall be taken as :

(1) For road bridges, $q_p(z)$ given in Table 3.9 shall be adopted, but the combination value $\psi_0 F_{wk}$ of the wind action on the bridge and on the vehicles travelling on the bridge, which is discussed in BS EN 1991-1-4 Clause 8.1(4), should be limited to a value F_W* determined by taking $q_p(z)$ as q'(z) given in Table 3.9. Provision given in the UK NA to BS EN 1991-1-4 Clause NA.2.47 shall not be followed.

(2) For railway underbridges, the value of $q_p(z)$ for determining F_W^{**}, which is discussed in BS EN 1991-1-4 Clause 8.1(5), shall be agreed with the appropriate railway authority taking into account the possibility of the presence of railway traffic on the bridge at high wind velocity.

3.4.3.4 Velocity Pressure on Relieving Area for Traffic Leading Combinations

Where wind on any part of a bridge or element gives relief to the member under consideration, the effective coexistent value of velocity pressure $q_L'(z)$ on the parts affording relief shall be taken as:

$$q_L'(z) = 1.2\, q'(z) / q_p(z)$$

where q'(z) and $q_p(z)$ are obtained from Table 3.9 appropriate to the height of the bridge and the loaded length under consideration.

Table 3.9 – Peak Velocity Pressure $q_p(z)$ and Hourly Mean Velocity Pressure $q'(z)$ for Full Procedure

Height z above ground level (m)	Peak Velocity Pressure $q_p(z)$ (kN/m^2) appropriate to horizontal wind loaded lengths (m)							Hourly Mean Velocity Pressure q'(z) (kN/m^2)
	20	100	200	400	600	1000	2000	
10	4.2	2.8	2.4	2.1	2.0	1.8	1.6	0.8
15	4.2	2.8	2.5	2.2	2.0	1.9	1.7	0.9
20	4.2	2.8	2.5	2.2	2.1	2.0	1.8	1.0
30	4.2	2.9	2.6	2.3	2.3	2.1	1.9	1.1
40	4.2	3.0	2.7	2.5	2.3	2.2	2.0	1.3
50	4.2	3.1	2.8	2.5	2.4	2.3	2.1	1.4
60	4.3	3.1	2.9	2.6	2.5	2.4	2.2	1.5
80	4.3	3.3	3.0	2.8	2.7	2.5	2.4	1.7
100	4.4	3.4	3.1	2.9	2.8	2.7	2.5	1.8
150	4.6	3.6	3.4	3.2	3.1	3.0	2.8	2.1
200	4.8	3.8	3.6	3.4	3.3	3.2	3.0	2.3

Notes:

(1) For locations which are less exposed, as described in Table 3.8 according to the degree of exposure, the values $q_p(z)$ and q'(z) given above may be factored according to the degree of exposure as follows :

Degree of exposure	Factor of $q_p(z)$ and q'(z)
1	0.7
2	0.8
3	0.9
4	1.0

(2) The horizontal wind loaded length shall be that giving the most severe effect. Where there is only one adverse area (see BS EN 1991-2) for the element or structure under consideration, the wind loaded length is the base length of the adverse area. Where there are more than one adverse area, as for continuous construction, the maximum effect shall be determined by consideration of any one adverse area or a combination of adverse areas, using the $q_p(z)$ appropriate to the base length of the total combined base lengths. The remaining adverse areas, if any, and the relieving areas, are subjected to wind having a velocity pressure as given in Clause 3.4.3.2 for wind leading combinations and in Clause 3.4.3.4 for traffic leading combinations.

(3) Where the bridge is located at an orography significant site, the $q_p(z)$ and q'(z) shall be factored by $(c_o(z))^2$ where $c_o(z)$ is a topographical factor. For the definition of significant orography, Clause 4.3.3 of BS EN 1991-1-4 and Clause NA.2.13 of the UK NA to BS EN 1991-1-4 shall be referred. $c_o(z)$ for q'(z) shall be determined based on Annex A.3 of BS EN 1991-1-4, and $c_o(z)$ for $q_p(z)$ shall be taken as follows:

$c_o(z) = 1$ for $\Phi < 0.05$

$$= 1 + 2s\phi \frac{s_c(z)}{s_b(z)} \quad \text{for } 0.05 < \Phi < 0.3$$

$$= 1 + 0.6s \frac{s_c(z)}{s_b(z)} \quad \text{for } \Phi > 0.3$$

where

s = orographic location factor as given in Annex A.3 of BS EN 1991-1-4
Φ = upwind slope as defined in Annex A.3 of BS EN 1991-1-4
$s_b(z)$ = terrain and bridge factor as given in Table 3.10
$s_c(z)$ = hourly velocity factor as given in Table 3.10

(4) Vertical elements such as piers and towers shall be divided into strips in accordance with the heights given in column 1 of this table and the $q_p(z)$ shall be derived from the centroid of each unit.

Table 3.10 – Terrain and Bridge Factor $s_b(z)$ and Hourly Velocity Factor $s_c(z)$

Height z above ground level (m)	Terrain and Bridge Factor $s_b(z)$									Hourly Velocity Factor $s_c(z)$
	Loaded Length (m)									
	20	**40**	**60**	**100**	**200**	**400**	**600**	**1000**	**2000**	
10	2.36	2.15	2.05	1.93	1.79	1.68	1.62	1.55	1.46	1.00
15	2.36	2.15	2.05	1.93	1.81	1.70	1.65	1.58	1.51	1.08
20	2.36	2.15	2.05	1.94	1.82	1.73	1.67	1.61	1.54	1.14
30	2.36	2.16	2.07	1.97	1.86	1.77	1.72	1.67	1.60	1.23
40	2.36	2.17	2.09	2.00	1.89	1.81	1.76	1.71	1.65	1.30
50	2.37	2.19	2.11	2.02	1.92	1.84	1.80	1.75	1.69	1.36
60	2.38	2.21	2.13	2.04	1.95	1.87	1.83	1.78	1.72	1.41
80	2.40	2.24	2.16	2.08	2.00	1.92	1.88	1.84	1.78	1.48
100	2.42	2.27	2.20	2.12	2.04	1.97	1.93	1.89	1.84	1.55
150	2.47	2.33	2.27	2.20	2.12	2.06	2.02	1.98	1.94	1.67
200	2.52	2.39	2.33	2.26	2.19	2.13	2.10	2.06	2.01	1.77
250	2.56	2.44	2.38	2.32	2.25	2.19	2.16	2.12	2.08	1.84
300	2.60	2.48	2.42	2.36	2.30	2.24	2.21	2.18	2.14	1.91
Gust Duration t (s)	3	4.8	6.3	9.0	14.5	23.4	31.0	44.1	71.1	

Note: Intermediate values may be interpolated using the relationships:

$$s_b(z) = \left\{1 + \left(\frac{z}{10}\right)^{-0.4} \left[1.48 - 0.704 \ln(\ln(t))\right]\right\} \left(\frac{z}{10}\right)^{0.19}$$ and

$$s_c(z) = \left(\frac{z}{10}\right)^{0.19}$$

where the gust duration, t in seconds, is related to the loaded length, L in metres, by the empirical relationship:

$t = 3$ for $L \leq 20$
$t = 0.375L^{0.69}$ for $L > 20$

When using the above formula for interpolating for short loaded lengths (< 50 m) and for height less than 25 m, z, the height above ground level in meters, shall be taken as 25 m.

3.4.3.5 Overturning Effects

(1) Where overturning effects are being investigated the wind load shall also be considered in combination with vertical traffic action. Where the vertical traffic action has a relieving effect, this load shall be limited to one notional lane or to one track only, and shall have the following value :

(a) on highway bridges, not more than 6 kN/m of bridge;

(b) on railway bridges, to be agree with the appropriate railway authority.

(2) For live load producing a relieving effect, the ULS partial factor γ_Q shall be taken as 1.0.

3.4.4 Dynamic Response Procedure

(1) Criteria given in Clause NA.2.49 of the UK NA to BS EN 1991-1-4 and PD 6688-1-4 Annex A for verifying the need of "dynamic response procedure" shall be followed. To account for the different wind environment in Hong Kong, the modifications given in Clauses 3.4.4.1 and 3.4.4.2 shall be made.

(2) Some background information on the aerodynamic effects on bridges is provided in Appendix D of this Manual for the designer's reference.

(3) When wind tunnel tests are required, the provisions given in Clause A.5 of PD 6688-1-4 and the further guidance on wind tunnel tests given in Appendix E of this Manual shall be followed.

3.4.4.1 General Substitutions

The site mean wind velocity $v_m(z)$ adopted in the UK NA Clause NA.2.49 and PD 6688-1-4 Annex A shall be replaced by the hourly mean wind velocity for relieving area $v_r(z)$, which shall be derived at the appropriate height of the bridge above ground level, z, in metres. The values for use in Hong Kong shall be taken as :

$v_r(z) = v_s\ s_c(z)\ c_o(z)$

where v_s is the hourly mean wind velocity at 10 m height and shall be taken as 35 m/s for all sites in Hong Kong. This is appropriate to a 120-year return period

$s_c(z)$ is the hourly velocity factor given in Table 3.10

$c_o(z)$ is the topographical factor given in Table 3.9.

3.4.4.2 Specific Substitutions

(1) The partial factors given in PD 6688-1-4 Clause A1.5.4.5 shall not be used, and shall be replaced by the partial factors γ_Q for wind actions given in Clause 3.2.1 of this Manual.

(2) The wind speed v_{WO} for verifying the divergent amplitude response required in PD 6688-1-4 Clauses A.2.4.2 and A.4.4.2, shall be taken as :

$$v_{WO} = K_{1U} K_{1A} \frac{[v_r(z) + 2v_d(z)]}{3}$$

where $v_d(z)$ is the peak wind velocity and shall be taken as :

$v_d(z) = v_s\ s_b(z)\ c_o(z)$

$v_r(z)$, v_s and $c_o(z)$ are defined in Clause 3.4.4.1

$s_b(z)$ is the terrain and bridge factor given in Table 3.10

K_{1U} is the factor to cover the uncertainty of prediction of wind velocity and shall be taken as 1.1

K_{1A} is the coefficient selected to give an appropriate low probability of occurrence of the severe forms of oscillation and shall be taken as 1.4.

(3) The wind speed criteria for wind tunnel testing required in PD 6688-1-4 Clause A.5, shall be taken as :

$$v_{WO} = K_{1U} K_{1A} \frac{[v_r(z) + 2v_d(z)]}{3}$$

$$v_{w\alpha} = K_{1U} K_{1A} v_r(z)$$

$$v_{WE} = K_{1U} K_{1A} \frac{[v_r(z) + v_d(z)]}{2}$$

where $v_r(z)$ is defined in Clause 3.4.4.1

$v_d(z)$, K_{1U} and K_{1A} are defined in Clause 3.4.4.2(2)

Angle of wind inclination $\alpha = \overline{\alpha} \pm 7\left[\frac{s_b(z)}{s_c(z)} - 1\right]$.

3.4.5 Wind Forces

(1) The simplified method given in Clause 8.3.2(1) of BS EN 1991-1-4 shall not be followed.

(2) The size factor c_s and the dynamic factor c_d, shall both be taken as 1.0 for bridges that a “dynamic response procedure” is not needed.

3.4.6 Force Coefficients

In addition to the force coefficients given in Clause 8.3.1 of BS EN 1991-1-4 and Table NA.8 of the UK NA to BS EN 1991-1-4, the force coefficients given in Clauses 3.4.6.1 to 3.4.6.5 below shall be followed.

3.4.6.1 *Truss Girder Superstructure without Traffic*

(1) For each windward truss, $c_{f,x}$ shall be determined from Table 3.11. The solidity ratio φ of the truss is the ratio of the net area to the overall area of the truss.

Table 3.11 – Force Coefficient $c_{f,x}$ for a Single Truss

Solidity Ratio φ	For Flatsided Members	For Round Members where d is diameter of member	
		$d*v_d(z) < 6m^2/s$	$d*v_d(z) \geq 6m^2/s$
0.1	1.9	1.2	0.7
0.2	1.8	1.2	0.8
0.3	1.7	1.2	0.8
0.4	1.7	1.1	0.8
0.5	1.6	1.1	0.8

(2) For the leeward truss of a superstructure with two trusses, the force coefficient $c_{f,x}$ given in Table 3.11 shall be multiplied by the shielding factor η obtained from Table 3.12. The spacing ratio in Table 3.12 is the distance between centres of trusses divided by the depth of the windward truss.

Table 3.12 – Shielding Factor η

Spacing Ratio	Value of η for solidity ratio φ of:				
	0.1	**0.2**	**0.3**	**0.4**	**0.5**
≤ 1	1.0	0.90	0.80	0.60	0.45
2	1.0	0.90	0.80	0.65	0.50
3	1.0	0.95	0.80	0.70	0.55
4	1.0	0.95	0.85	0.70	0.60
5	1.0	0.95	0.85	0.75	0.65
6	1.0	0.95	0.90	0.80	0.70

(3) Where a structure has more than two trusses, the force coefficient in transverse direction for the truss adjacent to the windward truss shall be derived as given in Clause 3.4.6.1(2) above. The coefficient for all other trusses shall be taken as equal to this value.

(4) For the deck construction, the force coefficient $c_{f,x}$ shall be taken as 1.1.

3.4.6.2 *Truss Girder Superstructures with Traffic*

The force coefficient $c_{f,x}$ for each truss and for the deck shall be the same as for the superstructure without traffic. $c_{f,x}$ for unshielded parts of the traffic shall be taken as 1.45.

3.4.6.3 Parapets

(1) For solid parapets the force coefficient $c_{f,x}$ shall be taken as equivalent to the $c_{f,x}$ for solid bridge deck as specified in BS EN 1991-1-4 Clause 8.3.1(1).

(2) For open parapets and safety fences, the $c_{f,x}$ shall be determined based on the shapes of the elements :

(a) Rectangular Elements – Refer to Clauses 7.6 and 7.13 of BS EN 1991-1-4 with the relevant recommendations given in the UK NA to BS EN 1991-1-4.

(b) Elements with Sharp Edged Sections – Refer to Clauses 7.7 and 7.13 of BS EN 1991-1-4 with relevant recommendations in the UK NA to BS EN 1991-1-4.

(c) Regular Polygonal Elements – Refer to Clauses 7.8 and 7.13 of BS EN 1991-1-4 with relevant recommendations in the UK NA to BS EN 1991-1-4.

(d) Circular Elements – Refer to Clauses 7.9.2 and 7.13 of BS EN 1991-1-4.

3.4.6.4 Sign Gantry

The force coefficient for the sign plate of sign gantry shall refer to Clause 7.4.3 of BS EN 1991-1-4, i.e. $c_{f,x}$ shall be taken as 1.8. For the gantry post, beam and other main elements, $c_{f,x}$ shall be obtained in accordance with Clause 3.4.6.3(2) above.

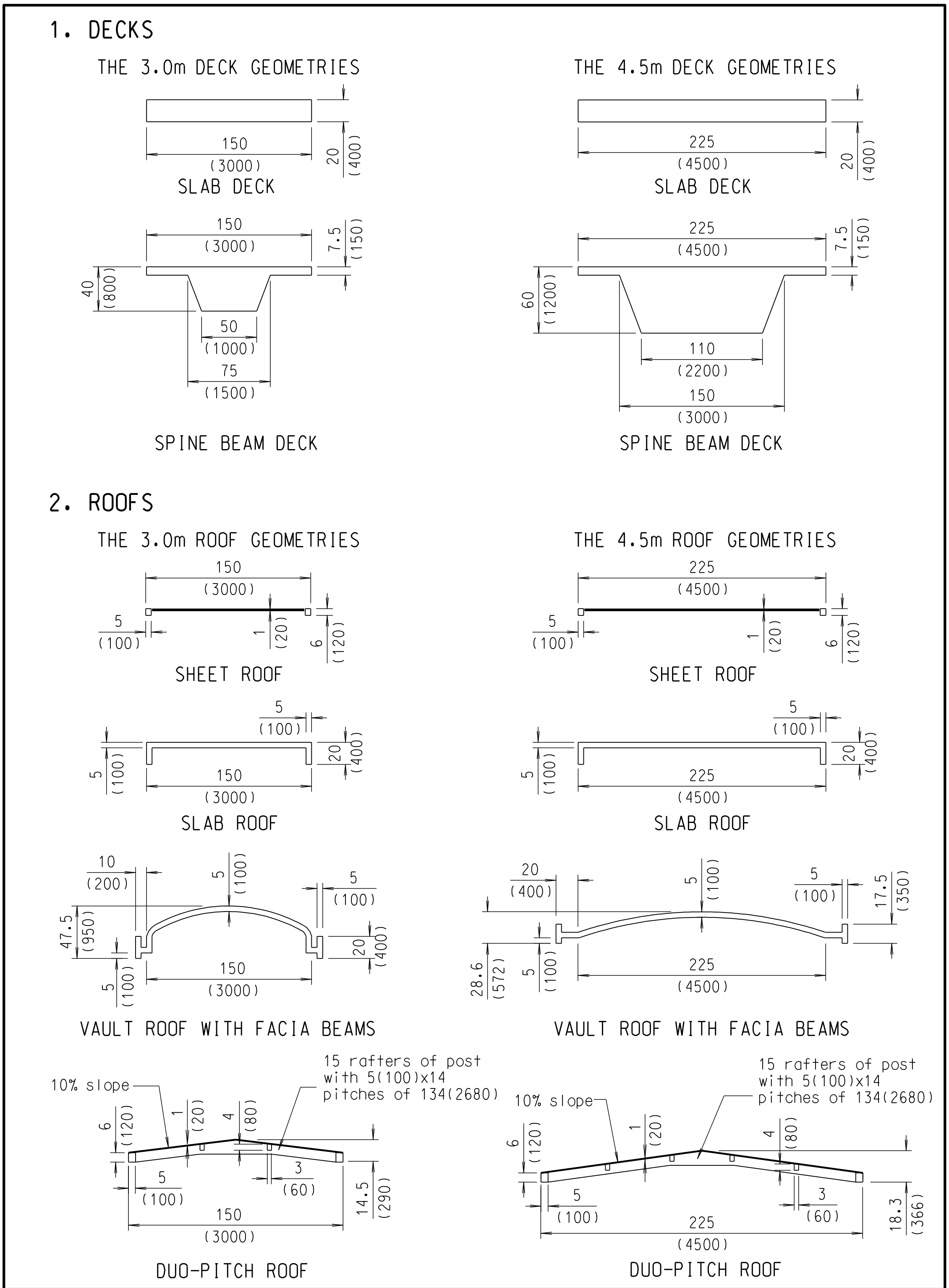
3.4.6.5 Covered Footbridges

(1) Hong Kong Special Administrative Region Government policy requires footbridges either to be covered, or to be designed so that covers can be added subsequently. The provisions for wind actions in BS EN 1991-1-4 and UK NA to BS EN 1991-1-4 are only applicable to uncovered bridges. The requirements given in this Section shall therefore be followed for covered footbridges.

(2) Wind tunnel tests have been carried out on sections of decks and roofs commonly adopted for covered footbridges in Hong Kong as shown in Figure 3.1 to determine suitable wind load coefficients for design purposes.

(3) Details of the test designs, and results of the tests, are given in the reports entitled "Aerodynamic Loads on Covered Footbridges" by British Maritime Technology. The reports include values of drag and lift coefficients for decks with roof and for roof only, covering the full range of designs at angles of wind inclination (α) varying between $\pm 20°$ to the horizontal.

Figure 3.1 - Typical Section of Decks and Roofs Modelled in Wind Tunnel Tests



Notes:
1. Dimensions model scale mm.
2. Equivalent full scale dimensions in brackets.
3. Both sheet roof and slab roof are classified as flat roof.

(4) Footbridges resembling the test designs as shown in Figure 3.1 shall be designed to resist wind actions derived from values of force coefficients $c_{f,x}$ and $c_{f,z}$ obtained from Table 3.13 and Table 3.14. The coefficient given in the tables are the most unfavourable values between wind inclination of ±5º because normal turbulence will cause wind inclination to vary between these angles. Where sidelong ground is concerned, the coefficients for angles of inclination corresponding to the fall of the ground shall be obtained from Table 3.15 and Table 3.16, and used as design values if greater than the coefficients for wind inclination varying between ±5º.

(5) For footbridges with shapes differing widely from the test designs, advice should be sought from aerodynamic specialists.

Table 3.13 – Force Coefficient $c_{f,x}$ for Covered Footbridges $-5^o \leq \alpha \leq +5^o$

Force Coefficient $c_{f,x}$	**for Deck with Roof**			**for Roof only**		
Deck Type \ **Roof Type**	**Flat Roof**	**Vault Roof**	**Duopitch Roof**	**Flat Roof**	**Vault Roof**	**Duopitch Roof**
Slab	2.1	1.6	1.8	1.8	0.9	0.9
Spine Beam Beam and Slab	1.8	1.6	1.6	2.1	1.1	1.1
Truss Girder	2.0	1.5	1.7	1.8	1.1	1.2
Slab, Spine Beam, Beam and Slab } with Solid Parapet Slab with Traffic Actions	1.7	1.3	1.4	2.4	1.5	1.7

Notes: (1) The transverse wind load on deck and parapet shall be the transverse wind load on deck with roof minus the transverse load on roof only.

(2) Refer to BS EN 1991-1-4 and UK NA to BS EN 1991-1-4 for calculation of transverse wind load.

Table 3.14 – Force Coefficient $c_{f,z}$ for Covered Footbridges $-5^o \leq \alpha \leq +5^o$

Force Coefficient $c_{f,z}$	**for Deck with Roof**			**for Deck only**			**for Roof only**		
Deck Type \ **Roof Type**	**Flat Roof**	**Vault Roof**	**Duo-pitch Roof**	**Flat Roof**	**Vault Roof**	**Duo-pitch Roof**	**Flat Roof**	**Vault Roof**	**Duo-pitch Roof**
Slab Truss Girder	+1.4 -0.7	+1.1 -0.6	+1.1 -0.6	-0.9	-0.8	+0.2 -1.0	+1.4 -0.2	+1.0	+1.2
Slab, Spine Beam, Beam and Slab } with Solid Parapet Slab with Traffic Actions	+1.2	+1.0	+1.1	+0.3 -0.1	+0.3 -0.1	+0.5 -0.1	+0.9	+0.7	+0.6
Spine Beam Beam and Slab	+2.1	+1.7	+2.0	+0.8	+0.7	+0.7 -0.1	+1.2	+0.9	+1.3

Notes: (1) Positive $c_{f,z}$ = upward wind load; negative $c_{f,z}$ = downward wind load.

(2) For deck-roof combination with both positive and negative $c_{f,z}$, both downward and upward wind load cases have to be considered. Otherwise, either upward (positive $c_{f,z}$) or downward (negative $c_{f,z}$) wind load is to be considered.

(3) Refer to BS EN 1991-1-4 and UK NA to BS EN 1991-1-4 for calculation of vertical wind load.

Table 3.15 – Force Coefficient $c_{f,x}$ for Covered Footbridges -20°<α<-5° & +5°<α<+20°

Force Coefficient $c_{f,x}$	for Deck with Roof			for Roof only		
Deck Type \ Roof Type	Flat Roof	Vault Roof	Duopitch Roof	Flat Roof	Vault Roof	Duopitch Roof
Slab	2.5	1.9	2.1	2.0	1	1.2
Spine Beam Beam and Slab	1.8	1.6	1.6	2.1	1.1	1.3
Truss Girder	2.0	--	--	1.8	--	--
Slab, Spine Beam, Beam and Slab } with Solid Parapet Slab with Traffic Actions	2.1	--	--	3.0	--	--

Notes: (1) The transverse wind load on deck and parapet shall be the transverse wind load on deck with roof minus the transverse load on roof only.
(2) Refer to BS EN 1991-1-4 and UK NA to BS EN 1991-1-4 for calculation of transverse wind load.
-- Test Results not available.

Table 3.16 – Force Coefficient $c_{f,z}$ for Covered Footbridges -20°<α<-5° & +5°<α<+20°

Force Coefficient $c_{f,z}$	for Deck with Roof			for Deck only			for Roof only		
Deck Type \ Roof Type	Flat Roof	Vault Roof	Duo-pitch Roof	Flat Roof	Vault Roof	Duo-pitch Roof	Flat Roof	Vault Roof	Duo-pitch Roof
Slab	-1.8 +2.1	-1.7 +2.1	-1.7 +2.0	-0.7 +2.0	-0.6 +0.9	-0.9 +1.0	-1.1 +1.2	-1.1 +1.2	-0.8 +1
Spine Beam Beam and Slab	-2.1 +1.8	-1.5 +1.8	-2.1 1.7	-1.2 +0.9	-1.1 +0.7	-1.3 +0.8	-1.1 +1.1	-1.1 +1.2	-0.8 +1.0
Truss Girder	-1.4 +1.7	-- --	-- --	+0.6 -0.7	-- --	-- --	-0.9 +0.7	-- --	-- --
Slab, Spine Beam, Beam and Slab } with Solid Parapet	-1.0	--	--	-0.1	--	--	-0.9	--	--
Slab with Traffic Actions	+1.8	--	--	+1.4	--	--	+0.4	--	--

Notes: (1) Positive $c_{f,z}$ = upward wind load; negative $c_{f,z}$ = downward wind load.
(2) Refer to BS EN 1991-1-4 and UK NA to BS EN 1991-1-4 for calculation of vertical wind load.
-- Test Results not available.

(6) A stairway model was also included among the wind tunnel tests. The stairway model test results indicate that the wind forces acting on a stairway may be greater than those acting on the adjacent main span. Values of

$c_{f,x}$ = 1.2 x force coefficient given in Table 3.13 and Table 3.15

$c_{f,z}$ = +1.7 or -1.1

$c_{f,y}$ = 2.35

shall accordingly be used for the design of stairways.

(7) Ramps will similarly experience wind forces greater than those acting on the adjacent main span. The values recommended above for stairways shall also be used for ramps.

(8) Where any additional wind tunnel tests are required, or any further guidance on interpretation or procedures for carrying out tests is required, the additional guidance given in Clause A.5 of PD 6688-1-4 and Appendix E of this Manual should be followed where appropriate.

3.4.7 Reference Area

(1) Provisions given in BS EN 1991-1-4 Clauses 8.3.1(4) and (5) on the reference area A_{ref} shall be followed.

(2) The $A_{ref,x}$ for load combination with traffic load should include the area of leeward parapet or noise barrier not screened by other members or traffic actions.

(3) For road bridges, a height of 2.5m from the level of the carriageway shall replace the height of 2m given in Clause 8.3.1(5)(a) of BS EN 1991-1-4.

(4) For footway/cycle track bridges, a height of 1.25m from the level of the finishing shall be used.

(5) For stairways and ramps, the plan area used to obtain the vertical wind load shall be the inclined area of the deck, not the projected area of the deck in plan.

3.4.8 ULS Partial Factors

(1) Appropriate ULS partial factor γ_Q for the design values of actions (EQU) (Set A), (STR/GEO) (Set B) and (STR/GEO) (Set C) shall be obtained in accordance with Clause 3.2.1 of this Manual. The ULS partial factors given in Tables A2.4(A) to A2.4(C) of BS EN 1990 and Tables NA.A2.4(A) to NA.A2.4(C) of the UK NA to BS EN 1990, shall not be used.

(2) For relieving effects of wind, γ_Q shall be taken as 1.0.

(3) The partial factors for wind actions associated with aerodynamic effects shall be obtained in accordance with Clause 3.2.1 of this Manual. The partial factors given in Clause A.1.5.4.5 of PD 6688-1-4 shall not be used.

3.4.9 ψ Factors

Appropriate ψ factor for combinations of actions shall be obtained in accordance with Clause 3.2.1 of this Manual. The ψ factors given in Tables A2.1 and A2.2 of BS EN 1990 and Tables NA.A2.1 and NA.A2.2 of the UK NA to BS EN 1990 shall not be used.

3.5 TEMPERATURE EFFECTS

3.5.1 General

(1) The provisions given in BS EN 1991-1-5 and the UK NA to BS EN 1991-1-5 for thermal actions on highway and railway structures shall be followed unless otherwise specified in this Manual.

(2) Daily and seasonal fluctuations in shade air temperature, solar radiation, re-radiation, etc. cause the followings :

(a) Changes in the uniform temperature of a bridge superstructure which, in turn, govern its movement. The uniform temperature is a theoretical temperature calculated by weighting and adding temperatures measured at various levels within the superstructure. The weighting is in the ratio of the area of cross-section at the various levels to the total area of cross-section of the superstructure. Over a period of time, there will be a minimum, a maximum, and a range of uniform bridge temperature, resulting in loads and/or load effects within the superstructure due to :

(i) restraint of associated expansion or contraction by the form of construction (e.g. portal frame, arch, flexible pier, elastomeric bearings) referred to as temperature restraint; and

(ii) friction at roller or sliding bearings where the form of the structure permits associated expansion and contraction, referred to as frictional bearing restraint.

(b) Differences in temperature between the top surface and other levels in the superstructure. These are referred to as temperature differences and they result in loads and/or load effects within the superstructure.

3.5.2 Uniform Temperature Components

(1) The minimum uniform bridge temperature $T_{e,min}$ and maximum uniform bridge temperature $T_{e,max}$ given in Figure 6.1 of BS EN 1991-1-5 and Clause NA.2.4 of the UK NA to BS EN 1991-1-5, and the minimum shade air temperature T_{min} and maximum shade air temperature T_{max} given in Clause NA.2.5 of the UK NA to BS EN 1991-1-5 shall not be used.

(2) The minimum and maximum uniform bridge temperatures ($T_{e,min}$ and $T_{e,max}$) for Hong Kong shall be obtained directly from Table 3.17 for superstructure Types 1 to 3. Basic uniform temperatures appropriate to a return period of 120 years shall be used except for the cases given in Clause 3.5.2(3).

Table 3.17 – Uniform Bridge Temperature

Superstructure Type (see Figure 3.2 for classification)	Return Period			
	120-Year		50-Year	
	$T_{e,min}$ °C	$T_{e,max}$ °C	$T_{e,min}$ °C	$T_{e,max}$ °C
1	0	46	0	44
2	0	40	0	38
3	0	36	0	34

(3) Basic uniform temperatures appropriate to a return period of 50 years may be used for :

(a) foot/cycle track bridges,

(b) carriageway joints and similar equipment likely to be replaced during the life of the structure, and

(c) erection loading.

(4) The uniform bridge temperatures are dependent on the depth of surfacing on the bridge deck, and the values given in Table 3.17 assume surfacing depths of 40 mm for Type 1 and 100 mm for Types 2 and 3. Where the depth of surfacing differs from these values, the minimum and maximum uniform bridge temperatures shall be adjusted by the amounts given in Table 3.18. Adjustments given in Table NA.1 of the UK NA to BS EN 1991-1-5 shall not be used.

Table 3.18 – Adjustment to Uniform Bridge Temperature for Deck Surfacing

	Additional To Minimum Uniform Bridge Temperature °C			Additional To Maximum Uniform Bridge Temperature °C		
Deck Surface	**Type 1**	**Type 2**	**Type 3**	**Type 1**	**Type 2**	**Type 3**
Unsurfaced Plain	0	-3	-1	+4	0	0
Unsurfaced Trafficked or Waterproofed	0	-3	-1	+2	+4	+2
40 mm Surfacing	0	-2	-1	0	+2	+1
100 mm Surfacing(1)	N/A	0	0	N/A	0	0
200 mm Surfacing(1)	N/A	+3	+1	N/A	-4	-2
Notes: (1) Surfacing depths include waterproofing (2) N/A = not applicable						

(5) The values of uniform temperature given in Table 3.17 shall be adjusted for height above mean sea level by subtracting 0.5ºC per 100 m height for minimum uniform temperatures and 1.0ºC per 100 m height for maximum uniform temperatures.

(6) The initial bridge temperature T_0 at the time the structure is effectively restrained upon completion of construction shall be taken as 30ºC for calculating contraction down to

the minimum uniform bridge temperature component and 10ºC for calculating expansion up to the maximum uniform bridge temperature component. Values given in Clause A.1(3) NOTE of BS EN 1991-1-5 and Clause NA.2.21 of the UK NA to BS EN 1991-1-5 shall not be used.

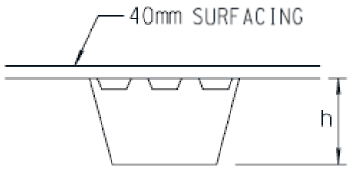
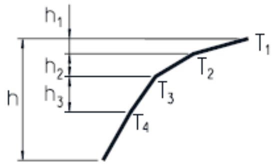
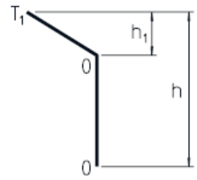
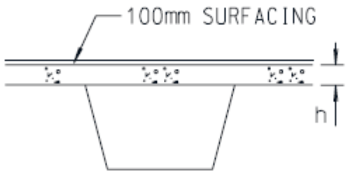
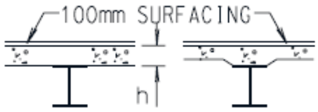
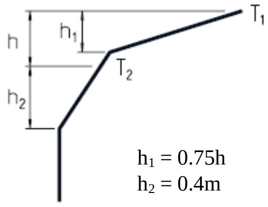
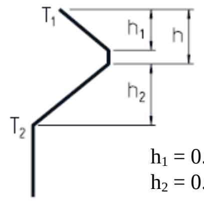
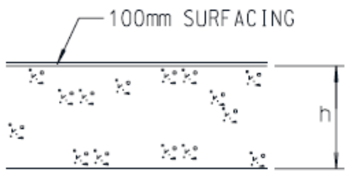
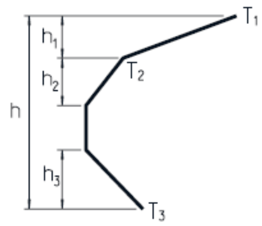
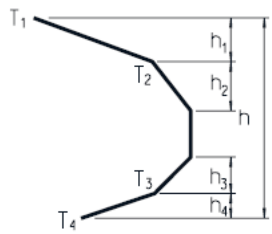
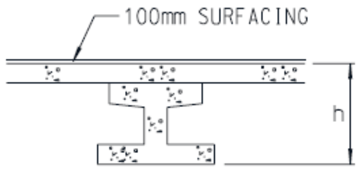
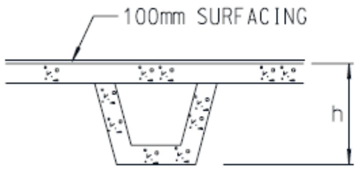
3.5.3 Temperature Difference Components

(1) The effects of the vertical temperature differences within the superstructure shall be considered by including a non-linear temperature difference component, i.e. Approach 2 defined in Clause 6.1.4.2 of BS EN 1991-1-5. The value of the sum of the linear and non-linear part of the temperature difference, ΔT, shall be determined from the data given in Figure 3.2. Figures 6.2a to 6.2c of BS EN 1991-1-5 and Clause NA.2.9 of the UK NA to BS EN 1991-1-5 shall not be used.

(2) Positive temperature differences occur when conditions are such that solar radiation and other effects cause a gain in heat through the top surface of the superstructure. Conversely, reverse temperature differences occur when conditions are such that heat is lost from the top surface of the bridge deck as a result of re-radiation and other effects.

(3) Temperature differences are sensitive to the thickness of surfacing, and the data given in Figure 3.2 assume depths of 40 mm for Type 1 and 100 mm for Type 2 and 3. For other depths of surfacing, the values given in Table 3.19, Table 3.20 and Table 3.21 may be used as appropriate. Temperature differences given in Annex B of BS EN 1991-1-5 and Clause NA.2.23 of the UK NA to BS EN 1991-1-5 shall not be used.

Figure 3.2 – Temperature Differences for Different Types of Superstructure

Group	Type of Superstructure	Temperature Difference (ΔT) (a) Heating	Temperature Difference (ΔT) (b) Cooling
1a	Steel deck on steel girders 	 h_1 = 0.1m, h_2 = 0.2m, h_3 = 0.3m ΔT_1 = 33ºC ΔT_2 = 19ºC ΔT_3 = 11ºC ΔT_4 = 6ºC	 ΔT_1 = 3ºC, h_1 = 0.5m
1b	Steel deck on steel truss or plate girders	See differences as for Group 1a	See differences as for Group 1a
2	Concrete deck on steel box, truss or plate girders  	 h_1 = 0.75h h_2 = 0.4m (see table 2(a) below)	 h_1 = 0.6h h_2 = 0.4m (see table 2(b) below)
3a	Concrete slab 		
3b	Concrete beams 	$h_1 = 0.4h \leq 0.15$m $h_2 = 0.4h \geq 0.08$m ≤ 0.25m $h_3 = 0.1h$ + surfacing depth in metres (For thin slabs, h_3 is limited by $h-h_1-h_2$)	$h_1 = h_4 = 0.20h \leq 0.25$m $h_2 = 0.25h \leq 0.4$m $h_3 = 0.15h \leq 0.4$m
3c	Concrete box girder 	(see table 3(a) below)	(see table 3(b) below)
Note:	The temperature difference ΔT incorporates ΔT_M and ΔT_E (see Clause 4(3) of BS EN 1991-1-5) together with a small part of component ΔT_U : this latter part has been included in the uniform bridge temperature component (see Clause 6.1.3 of BS EN 1991-1-5 and Clause 3.5.2 of this Manual).		

Group 2 – (a) Heating:

h	ΔT_1	ΔT_2
m	ºC	ºC
0.2	19	11
0.3	19	5

Group 2 – (b) Cooling:

h	ΔT_1	ΔT_2
m	ºC	ºC
0.2	1.0	9.0
0.3	4.0	9.0

Groups 3a–3c – (a) Heating:

h	ΔT_1	ΔT_2	ΔT_3
m	ºC	ºC	ºC
≤0.2	12.3	5.0	0.0
0.3	15.5	5.5	0.0
0.4	16.9	5.5	0.0
0.7	17.7	6.7	0.0
1.0	17.9	6.7	0.2
≥3.0	18.7	7.0	0.9

Groups 3a–3c – (b) Cooling:

h	ΔT_1	ΔT_2	ΔT_3	ΔT_4
m	ºC	ºC	ºC	ºC
≤0.2	1.8	0.8	0.3	0.9
0.3	2.9	1.2	0.4	1.6
0.4	3.7	1.3	0.7	2.3
0.7	6.8	2.3	1.5	4.6
1.0	9.1	2.9	2.2	6.7
≥3.0	11.3	4.1	3.5	8.9

Table 3.19 – Values of ΔT for Superstructure Type 1

Surfacing Thickness	Temperature Difference				
	Heating				Cooling
	ΔT_1	ΔT_2	ΔT_3	ΔT_4	ΔT_1
mm	ºC	ºC	ºC	ºC	ºC
unsurfaced	39	21	8	4	5
20	36	20	12	7	4
40	33	19	11	6	3

Table 3.20 – Values of ΔT for Superstructure Type 2

Depth of Slab (h)	Surfacing Thickness	Temperature Difference			
		Heating		Cooling	
		ΔT_1	ΔT_2	ΔT_1	ΔT_2
m	mm	ºC	ºC	ºC	ºC
0.2	U.P.	17	7	5	9
	U.T.	26	12	5	9
	waterproofed	30	14	5	9
	50	24	12	3	9
	100	19	11	1	9
	150	16	10	1	9
	200	13	9	1	9
0.3	U.P.	17	3	7	9
	U.T.	26	5	7	9
	waterproofed	30	7	7	9
	50	24	6	5	9
	100	19	5	4	9
	150	16	5	3	10
	200	13	5	2	11

Notes: (1) U.P. = unsurfaced plain
(2) U.T. = unsurfaced trafficked

Table 3.21 – Values of ΔT for Superstructure Type 3

Depth of Slab (h)	Surfacing Thickness	Temperature Difference						
		Heating			Cooling			
		ΔT_1	ΔT_2	ΔT_3	ΔT_1	ΔT_2	ΔT_3	ΔT_4
m	mm	ºC	ºC	ºC	ºC	ºC	ºC	ºC
≤ 0.2	U.P.	13.3	6.6	-	5.5	2.1	0.1	1.0
	U.T.	19.3	7.4	-	5.5	2.1	0.1	1.0
	waterproofed	21.7	8.9	-	5.5	2.1	0.1	1.0
	50	16.2	5.0	-	3.1	1.6	0.2	0.7
	100	12.3	5.0	-	1.8	0.8	0.3	0.9
	150	9.5	4.0	-	1.0	0.3	0.3	0.8
	200	7.4	3.3	-	1.0	0.3	0.3	0.8
0.3	U.P.	16.1	5.3	-	6.7	3.1	0.2	1.3
	U.T.	23.6	7.8	-	6.7	3.1	0.2	1.3
	waterproofed	26.6	9.0	-	6.7	3.1	0.2	1.3
	50	20.2	7.1	-	4.4	2.0	0.3	1.3
	100	15.5	5.5	-	2.9	1.2	0.4	1.6
	150	12.0	4.2	-	1.8	0.6	0.7	1.9
	200	9.3	3.5	-	1.0	0.2	0.8	1.9
0.4	U.P.	17.2	5.2	-	7.6	3.5	0.3	1.8
	U.T.	25.2	8.1	-	7.6	3.5	0.3	1.8
	waterproofed	28.4	9.2	-	7.6	3.5	0.3	1.8
	50	21.8	7.3	-	5.3	2.2	0.5	2.1
	100	16.9	5.5	-	3.7	1.3	0.7	2.3
	150	13.1	4.5	-	2.5	0.8	0.9	2.5
	200	10.1	3.6	-	1.7	0.4	1.2	2.8
0.7	U.P.	17.7	6.2	-	10.6	4.3	0.9	3.7
	U.T.	25.9	9.1	-	10.6	4.3	0.9	3.7
	waterproofed	28.4	10.4	-	10.6	4.3	0.9	3.7
	50	21.8	8.2	-	8.6	3.2	1.2	4.1
	100	16.9	6.7	-	6.8	2.3	1.5	4.6
	150	13.1	5.3	-	5.3	1.7	1.7	5.0
	200	10.1	4.1	-	4.1	1.2	2.1	5.3
1.0	U.P.	18.0	6.3	-	13.5	4.7	1.7	6.0
	U.T.	26.2	9.4	-	13.5	4.7	1.7	6.0
	waterproofed	29.5	10.3	-	13.5	4.7	1.7	6.0
	50	23.1	8.3	-	11.1	3.7	1.9	6.3
	100	17.9	6.7	0.2	9.1	2.9	2.2	6.7
	150	13.8	5.1	0.2	7.4	2.2	2.4	6.9
	200	10.7	4.1	0.2	5.8	1.7	2.6	7.2
≥ 3.0	U.P.	19.1	6.7	0.8	16.5	6.2	3.5	8.9
	U.T.	27.5	9.8	0.6	16.5	6.2	3.5	8.9
	waterproofed	30.9	11.1	0.5	16.5	6.2	3.5	8.9
	50	24.1	8.6	0.9	13.7	5.0	3.5	8.9
	100	18.7	7.0	0.9	11.3	4.1	3.5	8.9
	150	14.4	5.5	0.9	9.3	3.3	3.5	8.9
	200	11.2	4.4	0.8	7.6	2.6	3.5	8.9

Notes: (1) U.P. = unsurfaced plain
(2) U.T. = unsurfaced trafficked

3.5.4 Simultaneity of Uniform and Temperature Difference Components

The values of ω_N and ω_M given in Clause 6.1.5 NOTE 1 of BS EN 1991-1-5 shall not be used. ω_N and ω_M shall be taken as 1.0. The ranges of uniform bridge temperature component for expansion and contraction ($\Delta T_{N,exp}$ and $\Delta T_{N,con}$) shall be determined from the relevant value of T_0 as defined in Clause 3.5.2(6) and the relevant values of $T_{e,max}$ and $T_{e,min}$ as defined in Clause 3.5.2.

3.5.5 Coefficient of Thermal Expansion

For the purpose of calculating temperature effects, the coefficients of thermal expansion shall be taken as 12×10^{-6} per ºC for structural steel and 10×10^{-6} per ºC for concrete.

3.5.6 ULS Partial Factors

Appropriate ULS partial factor γ_Q for the design values of actions (EQU) (Set A), (STR/GEO) (Set B) and (STR/GEO) (Set C) shall be obtained in accordance with Clause 3.2.1 of this Manual. The ULS partial factors given in Tables A2.4(A) to A2.4(C) of BS EN 1990 and Tables NA.A2.4(A) to NA.A2.4(C) of the UK NA to BS EN 1990 shall not be used.

3.5.7 ψ Factors

Appropriate ψ factor for combinations of actions shall be obtained in accordance with Clause 3.2.1 of this Manual. The ψ factors given in Tables A2.1 and A2.2 of BS EN 1990 and Tables NA.A2.1 and NA.A2.2 of the UK NA to BS EN 1990 shall not be used.

3.6 ACCIDENTAL ACTIONS

3.6.1 General

(1) Structures should be designed and constructed so that it is inherently robust and not unreasonably susceptible to the effects of accidents or misuse, and disproportionate collapse.

(2) The overall structural integrity of the bridge shall be maintained following impact from road vehicles, derailed trains or ships on the bridge, but local damage to a part of the bridge support or deck can be accepted.

(3) In applying these requirements, checks shall be made for overall stability, local effects and progressive failure after removing elements whose load bearing capacity would be directly impaired as appropriate. In particular, for bridge decks :

   (a) The bridge deck must not lift or slide off its bearings.

(b) In the case of bridge decks with a number of carrying members e.g. beam and slab type decks, the structure as a whole must not collapse with any one of the carrying members being assumed to have failed; alternatively individual members can be checked for failure as at (c).

(c) In the case of bridge decks with a single carrying member e.g. spine beams, local failure or damage of elements (e.g. webs or flanges) or of joints between elements may be allowed but the structure as a whole must not collapse.

(4) For bridge decks with a small number of beams or girders, the designer may choose to include the reduced contribution of an individual damaged beam rather than assume it to be ineffective. This is also applicable to parts of voided slabs.

(5) The applicability of the various checks to different types of bridge decks is described in Table 3.22.

Table 3.22 – Application of Various Checks to Different Types of Bridge Decks

Type of Deck	**Check for overall stability**	**Check for progressive failure after removing elements whose load bearing capacity would be directly impaired**	**Check load effects**
Slab	Applicable	Not applicable	Not applicable
Voided Slab	Applicable	Applicable. Remove portion of web and/or flange which may be rendered ineffective.	Not applicable in general but may be required optionally.
Beam and Slab or plate girders and slab	Applicable	Applicable. Remove beam or girder which may be struck (not necessarily the outer member).	Not applicable in general but may be required optionally.
Other types including spine beams or decks with small number of beams or cells	Applicable	Not applicable	Applicable

3.6.2 Accidental Actions Caused by Road Vehicles

3.6.2.1 General

(1) The provisions given in BS EN 1991-1-7, UK NA to BS EN 1991-1-7 and PD6688-1-7 for accidental actions due to impact on highway and railway structures shall be followed unless otherwise specified in this Manual.

(2) Actions due to impact shall be represented by the equivalent static forces. Dynamic analysis shall not be adopted unless designers could demonstrate the need to the satisfaction of the Chief Highway Engineer/Bridges and Structures.

3.6.2.2 Impact on Supporting Substructures

(1) Notwithstanding the requirements of this Section for the protection of supporting substructures, barrier fences shall be provided in accordance with the Transport Planning and Design Manual (TPDM) to keep damages and injuries to errant vehicles and their occupants to a minimum.

(2) All supports shall be capable of resisting the main and residual load components acting simultaneously. Forces in the direction of normal travel F_{dx} shall be considered separately from forces perpendicular to the direction of normal travel F_{dy}.

3.6.2.2.1 Risk ranking procedure for supports

(1) The risk ranking procedure specified in Clause NA.2.11.2.3 of the UK NA to BS EN 1991-1-7 for road bridge supports and Clause 2.5.1(c) of PD 6688-1-7 for foot/cycle track bridge support shall be followed to determine the risk ranking factor R_{de}.

(2) When determining the factors F_2 and F_8, the design values of annual average daily traffic (AADT) and percentage of heavy goods vehicles (HGVs) with three or more axles of the considered roads should be determined on a project-specific basis. The traffic flow data of the considered roads based on the annual traffic census of the Transport Department may also be referred. Where such information is not available, the AADT values given in Table 3.23 with the assumed proportion of HGVs with three or more axles taken as 10% may be used.

Table 3.23 – AADT Values for Factor F_2 and F_8

Carriageway Type	AADT
1 lane	15,000
2 lane	33,333
3 lane	58,333
4 lane	75,000
Dual 1 lane	30,000
Dual 2 lane	66,666
Dual 3 lane	116,667
Dual 4 lane	150,000

(3) The risk ranking procedure for gantry supports should be the same as that for road bridge supports, except that factor F_7 for deck stability should be taken as 2.0, while factor F_8 should be determined with $AADT_{over}$ taken as 0.

3.6.2.2.2 Impact on road bridge supports

(1) The equivalent static design forces due to vehicular impact on road bridge supports and the associated adjustment factor F_a shall be determined in accordance with Table 3.24. The design forces and adjustment factor F_a given in Table NA.1 and Clause NA.2.11.2.4 of the UK NA to BS EN 1991-1-7 respectively shall not be used.

Table 3.24 –Equivalent Static Design Forces due to Vehicular Impact on Members Supporting Road Bridges over or adjacent to Roads

	Force F_{dx} in the direction of normal travel (kN)	**Force F_{dy} perpendicular to the direction of normal travel (kN)**	**Point of application on bridge support**
For all road bridges			
Main load component	1650	825	At the most severe point between 0.75m and 1.5m above carriageway level
Residual load component	825	415	At the most severe point between 1m and 3m above carriageway level
Minimum forces for robustness			
Main load component	250	250	At the most severe point between 0.75m and 1.5m above carriageway level
Residual load component	165	165	At the most severe point between 1m and 3m above carriageway level

Notes: (1) The adjustment factor F_a shall be applied to all main and residual load components except for the minimum forces for robustness.

(2) The adjustment factor F_a shall be taken as:

$F_a = 1.0$ for bridges over roads where speed limit < 72kph
$F_a = 1.0$ for bridges over roads where speed limit ≥ 72kph and $R_{de} \leq T_a = 2.4$
$F_a = 2.0$ for bridges over roads where speed limit ≥ 72kph and $R_{de} > T_a = 2.4$
$F_a = 1.0$ for bridges over roads where speed limit ≥ 72kph and $R_{de} > T_a = 2.4$ with supports protected by L3 containment level barriers with full working width

(2) When L4 containment level barriers with full working width are provided to protect the bridge supports, the main and residual load components of the minimum forces for robustness should be adopted.

(3) When L4 containment level concrete rigid barriers with a minimum lateral clearance of 400 mm between the barrier and the bridge support are provided, the design static forces should be taken as either only a residual load component specified for road bridges or the main and residual load components of minimum forces for robustness, whichever gives the most unfavourable effect on the structure. Provisions given in Clause 2.7 of PD 6688-1-7 shall not be used.

3.6.2.2.3 Impact on foot/cycle track bridge supports

(1) The equivalent static design forces due to vehicular impact on foot/cycle track bridge supports and the associated adjustment factor F_a shall be determined in accordance with Table 3.25. The design forces given in Table 1 of the PD 6688-1-7, and the provisions and adjustment factor given in Clause 2.5.1(d) of PD 6688-1-7 shall not be used.

Table 3.25 –Equivalent Static Design Forces due to Vehicular Impact on Members Supporting Foot/Cycle Track Bridges over or adjacent to Roads

	Force F_{dx} in the direction of normal travel (kN)	**Force F_{dy} perpendicular to the direction of normal travel (kN)**	**Point of application on bridge support**
For foot/cycle track bridges over roads where speed limit ≥ 72kph			
Main load component	1650	825	At the most severe point between 0.75m and 1.5m above carriageway level
Residual load component	825	415	At the most severe point between 1m and 3m above carriageway level
For foot/cycle track bridges over roads where speed limit < 72kph			
Main load component	825	415	At the most severe point between 0.75m and 1.5m above carriageway level
Residual load component	415	205	At the most severe point between 1m and 3m above carriageway level
Minimum forces for robustness			
Main load component	165	165	At the most severe point between 0.75m and 1.5m above carriageway level
Residual load component	85	85	At the most severe point between 1m and 3m above carriageway level

Notes: (1) The adjustment factor F_a shall be applied to all main and residual load components except for the minimum forces for robustness.

(2) The adjustment factor F_a shall be taken as:

$F_a = 0.5$ for bridges with $R_{de} \leq T_c = 2.4$
$F_a = 1.0$ for bridges with $R_{de} > T_c = 2.4$
$F_a = 0.5$ for bridges over roads where speed limit ≥ 72kph and $R_{de} > T_c = 2.4$ with supports protected by L3 containment level barriers with full working width

(2) If the risk ranking factor R_{de} is larger than $T_c = 2.4$ and when L4 containment level barriers with full working width are provided to protect the bridge supports, the main and residual load components of the minimum forces for robustness should be adopted. When L4 containment level concrete rigid barriers with a minimum lateral clearance of 400 mm between the barrier and the bridge support are provided, the static design forces should be taken as either only a residual load component of $F_{dx} = F_{dy} = 165$ kN or the main and residual load components of minimum forces for robustness, whichever

gives the most unfavourable effect on the structure. Provisions given in Clause 2.5.1(f) of PD 6688-1-7 shall not be used.

(3) If the risk ranking factor R_{de} is less than or equal to T_c = 2.4 and when L3 containment level barriers with full working width are provided to protect the bridge supports, the main and residual load components of the minimum forces for robustness should be adopted. When L3 containment level concrete rigid barriers with a minimum lateral clearance of 400 mm between the barrier and the support of foot/cycle track bridges are provided, the static design forces should be taken as either only a residual load component of $F_{dx} = F_{dy}$ = 85 kN or the main and residual load components of minimum forces for robustness, whichever gives the most unfavourable effect on the structure. Provisions given in Clause 2.5.1(f) of PD 6688-1-7 shall not be used.

3.6.2.2.4 Impact on gantry supports

The equivalent static design forces due to vehicular impact on gantry supports and the associated adjustment factor F_a shall be determined in accordance with Table 3.26.

Table 3.26 – Equivalent Static Design Forces due to Vehicular Impact on Members Supporting Sign Gantries over or adjacent to Roads

	Force F_{dx} in the direction of normal travel (kN)	**Force F_{dy} perpendicular to the direction of normal travel (kN)**	**Point of application on bridge support**
For gantries over roads where speed limit ≥ 72kph			
Main load component	1650	825	At the most severe point between 0.75m and 1.5m above carriageway level
Residual load component	825	415	At the most severe point between 1m and 3m above carriageway level
Minimum forces for robustness			
Main load component	165	165	At the most severe point between 0.75m and 1.5m above carriageway level
Residual load component	85	85	At the most severe point between 1m and 3m above carriageway level

Notes: (1) The adjustment factor F_a shall be applied to all main and residual load components except for the minimum forces for robustness.

(2) The adjustment factor F_a shall be taken as 0.2 for gantries with $R_{de} > 0.5$.

(3) The minimum forces for robustness shall be adopted for gantries over roads where speed limit < 72kph or $R_{de} \leq 0.5$.

3.6.2.3 Impact on Superstructures

(1) For road bridges, the equivalent static design forces due to vehicular impact on superstructures shall be determined in accordance with Table 3.27. The equivalent static design forces and the relevant heights for the reduction factor r_F given in Tables NA.9 to NA.10 and Clause NA.2.17 of the UK NA to BS EN 1991-1-7, shall not be used.

Table 3.27 – Equivalent Static Design Forces due to Vehicular Impact on Bridge Superstructures

Force F_{dx} in the direction of normal travel (kN)	**Force F_{dy} perpendicular to the direction of normal travel (kN)**	**Point of application on superstructure**
825	415	On vertical surface or underside surface of bridge deck in any direction between the horizontal and vertically upwards

Notes:

(1) For foot/cycle track bridges over roads where speed limit ≤ 72kph, F_{dx} and F_{dy} shall be taken as 410 kN and 205 kN respectively.

(2) Reduction factor r_F shall be determined in accordance with Figure 4.2 of BS EN 1991-1-7 with the h_0 and h_1 taken as 5.7 m and 6.0 m respectively after compensation for vertical curvature and deflection. (The maximum deflection of the structure should be calculated at the SLS using the frequent combination of actions.)

(3) Notwithstanding the provision in Note (2), adequate restraint to the bridge deck should be provided to prevent the deck from being removed from the support under the action of the vehicle collision forces given in this table in all cases.

(2) For sign gantries, consideration of vehicular impact on superstructures is not applicable and a minimum headroom of 5.5m shall be provided.

(3) The force in the direction of normal travel F_{dx} shall be considered separately from the force perpendicular to the direction of normal travel F_{dy}.

3.6.3 Accidental Actions Caused by Derailed Rail Traffic under or adjacent to Structures

(1) The potential accidental actions from a derailed train colliding with the substructure of a bridge crossing a railway track is very large. To design a support capable of successfully withstanding such a loading may be very difficult. Nevertheless, because of the potentially disastrous consequences, consideration shall always be given to ways of alleviating the effects of such a collision. The appropriate railway authority shall be consulted for the design of bridge substructures across or adjacent to railway tracks.

(2) The provisions given in BS EN 1991-1-7, UK NA to BS EN 1991-1-7 and PD6688-1-7 should also be followed as far as they are applicable to Hong Kong and if agreed by the appropriate railway authority.

(3) See also Clause 10.2 of this Manual for recommendations on means by which the severity of collision effects may be ameliorated.

3.6.4 Accidental Actions Caused by Ship Traffic

(1) Bridges over navigation channels may be subjected to accidental actions caused by ship traffic. The magnitude and form of such collision forces depend so much on the location of the bridge and the nature of the shipping using the navigation channel that specific guidance cannot be given, but the possibility of ship collisions shall always be considered at the design stage and appropriate protection provided.

(2) The provisions given in BS EN 1991-1-7, UK NA to BS EN 1991-1-7 and PD6688-1-7 should be followed as far as they are applicable to Hong Kong.

(3) Clauses 10.4.3 and 16.2 of this Manual also deal with ship collisions.

3.7 TRAFFIC ACTIONS

3.7.1 General

The provisions given in BS EN 1991-2, UK NA to BS EN 1991-2 and PD 6688-2 shall be followed unless otherwise specified in this Manual.

3.7.2 Actions on Road Bridges

3.7.2.1 Models of Road Traffic Loads

(1) Load effects due to road traffic on main routes in Hong Kong with a return period of 2400 years (or a probability of exceedence of 5% in the design life of 120 years) are used as the basis for the calibration of the main Load Models of the Eurocodes for use in Hong Kong.

(2) Where special considerations indicate that a lesser live load would be appropriate, the agreement of the Chief Highway Engineer/Bridges and Structures to its use must first be obtained.

3.7.2.2 Load Model 1

(1) The adjustment factors α_{qi} for the uniformly distributed load q_{ik} and α_{Qi} for the tandem system Q_{ik} of Load Model 1 (LM1) given in Clause 4.3.2 of BS EN 1991-2 shall be obtained from Table 3.28. The α_{qi} and α_{Qi} given in Table NA.1 of the UK NA to BS EN 1991-2 shall not be used.

Table 3.28 – Adjustment Factors α_{Qi} and α_{qi} for Load Model 1

Location	**α_{Qi} for tandem axle loads**	**α_{qi} for UDL loading**
Lane 1	α_{Q1} = 1.20	α_{q1} = 0.53
Lane 2	α_{Q2} = 1.00	α_{q2} = 1.91
Lane 3	α_{Q3} = 1.00	α_{q3} = 1.91
Other lanes	-	α_{qn} = 1.91
Remaining area	-	α_{qr} = 1.91
Where the loaded length is less than 60 m and N is greater than or equal to 6 then these should be modified to:		
Location	**α_{Qi} for tandem axle loads**	**α_{qi} for UDL loading**
Lane 1	α_{Q1} = 1.44	α_{q1} = 0.64
Lane 2	α_{Q2} = 1.20	α_{q2} = 2.30
Lane 3	α_{Q3} = 1.20	α_{q3} = 2.30
Other lanes	-	α_{qn} = 2.30
Remaining area	-	α_{qr} = 2.30
Notes: (1) α_{q1} shall be taken as 1.0 when determining the braking force. (2) The value of N is to be taken as the total number of notional lanes on the bridge (this shall included all the lanes for dual carriageway roads) expect that for a bridge carrying one way traffic only, the value of N shall be taken as twice the number of notional lanes on the bridges.		

(2) In general, the use of Load Model 1 defined in this Section is safe-sided for road bridges with loaded lengths over 200 m. Subject to the agreement of the Chief Highway Engineer/Bridges and Structures, project-specific load models may be used for loaded lengths over 200 m in individual project.

3.7.2.3 Load Model 2

The adjustment factor β_Q for the single axle load Q_{ak} of Load Model 2 (LM2) given in Clause 4.3.3 of BS EN 1991-2 shall be taken as 0.9. The β_Q given in Clause NA.2.14 of the UK NA to BS EN 1991-2 shall not be used.

3.7.2.4 Load Model 3

(1) Load Model 3 (LM3) shall be taken as the SV196 of the Special Types General Order (STGO) vehicle given in Clause NA.2.16.1.3 of the UK NA to BS EN 1991-2. The other STGO vehicles and Special Order Vehicles (SOV) given in Clauses NA.2.16.1 and NA.2.16.2 of the UK NA to BS EN 1991-2 may be used if agreed with the Chief Highway Engineer/Bridges and Structures.

(2) Load Model 3 may be disregarded for structures spanning less than 15m situated on rural roads other than trunk or main roads.

3.7.2.5 Load Model 4

The crowd loading of Load Model 4 (LM4), equivalent to 5 kN/m^2, need only be considered where there is a probability of such loading and so its use shall be agreed on a project-specific basis.

3.7.2.6 Horizontal Forces

(1) The adjustment factors α_{qi} and α_{Qi} of LM1 shall be determined in accordance with Clause 3.7.2.2 when determining the braking force Q_{lk} given in Clauses 4.4.1(1) to (2) of BS EN 1991-2. The α_{qi} and α_{Qi} given in Table NA.1 of the UK NA to BS EN 1991-2 shall not be used.

(2) The SV196 of the STGO vehicle shall be used to determine the horizontal forces associated with Load Model 3 given in Clause NA.2.18 of the UK NA to BS EN 1991-2.

3.7.2.7 Groups of Traffic Loads on Road Bridges

For the characteristic values of the multi-component actions, the simultaneity of the loading systems as defined in BS EN 1991-2 and the UK NA to BS EN 1991-2 with the modifications given in Clauses 3.7.2.2 to 3.7.2.6 and 3.7.3 shall be taken into account by considering the groups of loads defined in Table 3.29. The groups of traffic loads given in Table NA.3 of the UK NA to BS EN 1991-2 shall not be used.

Table 3.29 – Assessment of Groups Traffic Loads (Characteristic Value of the Multi-Component Actions)

Load Type		**Carriageway**						**Footways and cycle tracks**
		Vertical forces				**Horizontal forces**		**Vertical forces only**
Reference of BS EN 1991-2		**4.3.2**	**4.3.3**	Annex A	**4.3.5**	**4.4.1**	**4.4.2**	**5.3.2.1** Equation (5.1)
Load system		LM1 (TS and UDL)	LM2 (Single axle)	LM3 (Special vehicles)	LM4 (Crowd loading)	Braking and acceleration forces	Centrifugal and transverse forces	Uniformly distributed load
Group of loads	**gr1a**	Characteristic						0.8 times Characteristic
	gr1b		Characteristic					
	gr2	Frequent(2)				Characteristic	Characteristic	
	gr3(1)							Characteristic
	gr4				Characteristic			Characteristic
	gr5	Frequent(2)		Characteristic				
	gr6			Characteristic		Characteristic	Characteristic	
		Dominant component action (the group is sometimes designated by this component for convenience).						

(1) This group is irrelevant if gr4 is considered.
(2) The ψ_1 factors should be taken in accordance with Clause 3.2.1.

3.7.2.8 Fatigue Load Models

The provisions given in Clause 4.6 of BS EN 1991-2 and Clauses NA.2.22 to NA.2.27 of the UK NA to BS EN 1991-2 shall be followed.

3.7.2.9 Actions for Accidental Design Situations

3.7.2.9.1 Collision forces from vehicles under the bridge

The provisions given in BS EN 1991-1-7, the UK NA to BS EN 1991-1-7 and PD 6688-1-7 with modifications given in Clause 3.6 shall be followed.

3.7.2.9.2 Vehicle on footways and cycle tracks on road bridges

The adjustment factor $\alpha_{Q2} = 1.0$ given in Table NA.1 of the UK NA to BS EN 1991-2 shall be used for determining the accidental wheel load for footways and cycle tracks on road bridges given in Clause 4.7.3.1 of BS EN 1991-2.

3.7.2.9.3 Collision forces on kerbs

The adjustment factor $\alpha_{Q1} = 1.0$ given in Table NA.1 of the UK NA to BS EN 1991-2 shall be used for determining the collision loads on kerbs given in Clause 4.7.3.2 of BS EN 1991-2.

3.7.2.9.4 Collision forces on vehicle restraint systems

(1) Forces due to collision with vehicle restraint systems for determining global effects required in Clause 4.7.3.3(1) of BS EN 1991-2 shall be obtained from Table 3.30. Table NA.6 of the UK NA to BS EN 1991-2 shall not be used. The adjustment factors α_{qi} and α_{Qi} of LM1 acting simultaneously with these collision forces should be taken in accordance with Clause 3.7.2.2.

Table 3.30 – Forces due to Collision with Vehicle Restraint Systems for Determining Global Effects

Class	Transverse force (kN)	Longitudinal force (kN)	Vertical force (kN)	Examples of applications
A	100	-	-	L1 and L2 Metal Parapets
B	200	-	-	L1 and L2 Composite Parapets
C	400	100	175	L3 Metal Parapets
D	600	100	175	L3 Composite Parapets and L4 Reinforced Concrete Wall Parapets

(2) For determining local effects required in Clause 4.7.3.3(2) of BS EN 1991-2, the full accidental wheel/axle loads given in Clause 4.7.3.1 of BS EN 1991-2 shall be taken as acting simultaneously with the horizontal collision forces.

3.7.2.10 Actions on Pedestrian Parapets

For the design of the supporting structure, if the pedestrian parapets are adequately protected against vehicle collisions, the minimum load shall be taken as that corresponds to Class 3 pedestrian restraint system of BS 7818, i.e. a uniformly distributed load of 1.4 kN/m acting horizontally or vertically at the top of the pedestrian parapet and considered to act simultaneously with the uniformly distributed load given in Clause 3.7.3. Actions on pedestrian parapets specified in Clause NA.2.32 of the UK NA to BS EN 1991-2 and Clause 3.14 of PD 6688-2 shall not be used.

3.7.2.11 Load Models for Abutments and Walls Adjacent to Bridges

(1) The adjustment factor α_{Q1} for determining the forces on abutments and walls given in Clause NA.2.34 of the UK NA to BS EN 1991-2 shall be taken in accordance with Clause 3.7.2.2. The α_{Q1} given in Table NA.1 of the UK NA to BS EN 1991-2 shall not be used.

(2) The SV196 of the STGO vehicle as given in Clause NA.2.16 of the UK NA to BS EN 1991-2 shall be used to determine the forces on abutments and walls given in Clause NA.2.34 of the UK NA to BS EN 1991-2.

3.7.2.12 Dynamic Effects on Road Bridges

(1) Dynamic effects on highway bridges are usually deemed to be covered by the allowance for impact included in traffic actions. However, although such considerations may be sufficient structurally, the possibility of highway users being adversely affected shall also be considered. Complaints about the liveliness of highway structures have been made in Hong Kong as a result of the occupants of traffic stalled in one lane of a structure being subjected to oscillations caused by traffic moving in a neighbouring lane. Similar situations could recur at any time under the conditions prevailing in Hong Kong.

(2) Highway structures oscillate in sympathy with passing vehicles oscillating on their suspensions as a result of road surface irregularities. The worst oscillations occur when the natural frequency of a structure lies within the range of forcing frequencies imposed by passing traffic.

(3) Such forcing frequencies generally range between 2 Hz and 5 Hz. Highway structures shall accordingly be designed so that as far as possible their natural frequencies lie outside this range.

3.7.3 Actions on Footways, Cycle Tracks and Footbridges

3.7.3.1 Load Models – Uniformly Distributed Load

The provisions given in Clause NA.2.36 of the UK NA to BS EN 1991-2 for footways and cycle tracks on road bridges shall also be applicable to footbridges. Clause 5.3.2.1(2) NOTE of BS EN 1991-2 shall not be used.

3.7.3.2 Actions for Accidental Design Situations for Footbridges

For collision forces from vehicles under footbridges, the provisions given in BS EN 1991-1-7, the UK NA to BS EN 1991-1-7 and PD 6688-1-7 with modifications given in Clause 3.6 shall be followed.

3.7.3.3 Dynamic Models of Pedestrian Loads

(1) Bridge Class D given in Table NA.7 in UK NA to BS EN 1991-2, i.e. group size N = 16 for walking, N = 4 for jogging and crowd density $\rho = 1.5$ person/m^2, shall be used for all footbridges in Hong Kong.

(2) The reduction factor, γ, to allow for the unsynchronized combination of actions within groups and crowds given in Figure 2 of PD6688-2 shall be used instead of Figure NA.9 of the UK NA to BS EN 1991-2.

(3) When determining the serviceability limit for vertical acceleration given in Clause NA.2.44.6 of the UK NA to BS EN1991-2, factor k_4 shall be taken as not greater than 1.0 unless determined otherwise for individual project and agreed with the Chief Highway Engineer/Bridges and Structures.

(4) The lateral response due to crowd loading need not be considered if there are no significant lateral modes with frequencies below 1.5Hz. For footbridges with lateral frequencies between 0.5 Hz and 1.1 Hz, the serviceability limits for lateral response shall be determined in accordance with NA.2.44.7 of the UK NA to BS 1991-2. For footbridges with lateral frequencies outside this range, specialist advice shall be sought.

(5) The possibility of a group of pedestrians deliberately causing a footbridge to oscillate resonantly shall be borne in mind. Footbridge bearings shall be designed to allow for this possibility, and prestressed concrete beams shall be provided with sufficient untensioned reinforcement to resist a reversal of 10% of the static live load bending moment. Guides shall be provided where necessary to prevent any tendency for a superstructure to bounce off its bearings.

3.7.3.4 Live Load on Footbridge and Subway Covers

Covers shall be designed to withstand all the appropriate combinations of actions described in Clause 3.2. In addition, covers shall be designed to resist a uniformly distributed load of 0.5 kN/m^2, which shall be considered as a variable traffic action and combination with other variable actions can be ignored.

3.7.4 ULS Partial Factors

Appropriate ULS partial factor γ_Q for the design values of actions (EQU) (Set A), (STR/GEO) (Set B) and (STR/GEO) (Set C) shall be obtained in accordance with Clause 3.2.1 of this Manual. The ULS partial factors given in Tables A2.4(A) to A2.4(C) of BS EN 1990 and Tables NA.A2.4(A) to NA.A2.4(C) of the UK NA to BS EN 1990 shall not be used.

3.7.5 ψ Factors

Appropriate ψ factor for combinations of actions shall be obtained in accordance with Clause 3.2.1 of this Manual. The ψ factors given in Tables A2.1 and A2.2 of BS EN 1990 and Tables NA.A2.1 and NA.A2.2 of the UK NA to BS EN 1990 shall not be used.

3.8 SEISMIC ACTIONS

Provisions for seismic actions are given in Chapter 4 of this Manual.

CHAPTER 4 DESIGN FOR EARTHQUAKE RESISTANCE

4.1 GENERAL

(1) Highway structures and railway bridges shall be designed for earthquake resistance. The provisions given in BS EN 1998-1, BS EN 1998-2, the UK NAs to BS EN 1998-1 and BS EN 1998-2 and the recommendations in PD 6698 shall be followed except where modified by this Manual.

(2) The provisions in this Chapter should be taken as the minimum requirements. For special bridges (e.g. cable-stayed bridges, suspension bridges, arch bridges and bridges with span exceeding 150 m) and bridges of critical importance, consideration should be given to the specification of appropriate project-specific requirements and compliance criteria to achieve higher performance for repairable and minimum damage and the conduction of site-specific hazard analysis for establishing the design seismic actions.

4.2 REFERENCE RETURN PERIOD

The reference return period for no-collapse requirement, T_{NCR}, given in Table NA.1 of the UK NA to BS EN 1998-2 for Clause 2.1.(3)P of BS EN 1998-2 shall not be used. The reference return period, T_{NCR}, shall be taken as 475 years.

4.3 REFERENCE PEAK GROUND ACCELERATION

The reference peak ground acceleration on type A ground, a_{gR}, that corresponds to the reference return period, T_{NCR}, of 475 years shall be taken as 0.12g. The recommendation given in the Table NA.1 of the UK NA to BS EN 1998-1 for Clauses 3.2.1(1), (2) and (3) of BS EN 1998-1 which makes reference to the seismic contour map in PD 6698 shall not be followed.

4.4 IMPORTANCE CLASS

The classification of importance classes and values of importance factors, γ_I, to be adopted shall be established on a project-specific basis. Those given in Table NA.1 of the UK NA to BS EN 1998-2 for Clause 2.1(4)P and 2.1(6) of BS EN 1998-2 shall not be used. Recommendations on classification of importance classes, and corresponding values of importance factors, γ_I, are given in Table 4.1. See also Clause 4.1(2) above for special bridges and bridges of critical importance.

Table 4.1 – Importance Class and Importance Factor

Importance Class	Importance Factor	Relevant Highway Structures
Class I	1.0	All highway structures not under Importance Class II or III.
Class II	1.4	When any one of the following conditions are met : • on traffic sensitive routes (Red and Pink Routes); • on public transport sensitive routes; or • on expressway.
Class III	2.3	When any one of the following conditions are met : • any span length > 150 m; • on expressway with total length > 1000 m; or • critical for maintaining communications, especially in the immediate post-earthquake period (e.g. on sole access routes to hospital).
Note: The importance factors of 1.0, 1.4 and 2.3 correspond to return periods of 475, 1000 and 2500 years respectively.		

4.5 SEISMIC ACTIONS AS AN ACCIDENTAL ACTION

The relaxations given in Table NA.1 of the UK NA to BS EN 1998-2 with regard to Clause 2.2.2(5) of BS EN 1998-2 shall not be followed. Seismic actions shall not be considered as an accidental action and the requirements of Clause 2.2.2(3) and (4) are applicable.

4.6 SIMPLIFIED DESIGN CRITERIA FOR LOW SEISMICITY

Cases of low seismicity shall be as defined in Clause 3.2.1(4) NOTE of BS EN 1998-1. The recommendations given in Table NA.1 of the UK NA to BS EN 1998-1 for Clause 3.2.1(4) of BS EN 1998-1 and Table NA.1 of the UK NA to BS EN 1998-2 for Clause 2.3.7(1) of BS EN 1998-2 shall not be followed. Simplified design criteria are therefore not applicable for Hong Kong and are not established in this Manual.

4.7 SITE DEPENDENT ELASTIC RESPONSE SPECTRUM

(1) In the absence of site-specific hazard analysis, Type 2 elastic response spectrum as defined in Clause 3.2.2 of BS EN 1998-1 shall be adopted with ground type determined in accordance with Table 3.1 in Clause 3.1.2 of BS EN 1998-1.

(2) For special bridges and bridges of critical importance, and also for sites with special ground conditions, site-specific hazard and ground motion response analyses for establishing the design response spectra should be considered.

(3) For the purpose of the requirement of Clause 3.2.2.3 of BS EN 1998-2 on near source effects, all faults within 10 km horizontally of Hong Kong may be considered not active unless revealed otherwise by site-specific assessment.

4.8 SPATIAL VARIABILITY OF SEISMIC ACTIONS

The relaxation given in Table NA.1 of the UK NA to BS EN 1998-2 for Clause 3.3(1) of BS EN 1998-2 that spatial variability of ground motions need not be considered for bridges with continuous decks, where the supports are founded on approximately uniform soil of type A, B or C, shall not be followed. The provisions given in Clause 3.3 of BS EN 1998-2 shall be adopted for all ground types.

4.9 QUASI-PERMANENT VALUES OF MASS CORRESPONDING TO TRAFFIC ACTIONS

The values of $\psi_{2,1}$ factors for the accompanying traffic actions given in Clause 4.1.2(4)P NOTE of BS EN 1998-2 shall not be used and shall be replaced by the following :

(a) Road Bridges – $\psi_{2,1}$ = 0.2 for Load Model 1 (LM1)

(b) Footbridges – $\psi_{2,1}$ = 0.2 for uniformly distributed load, q_{fk}

(c) Railway underbridges – value of $\psi_{2,1}$ shall be agreed with the appropriate railway authority

4.10 STRENGTH VERIFICATIONS – MATERIALS

(1) The provision given in Clause 5.2.1(1)P of BS EN 1998-2 for requirement of reinforcing steel of Class C in accordance with EN 1992-1-1:2004 Table C.1, shall be replaced by reinforcing steel of Grade 500C in accordance with CS2:2012.

(2) The provision given in Clause 5.2.1(2) of BS EN 1998-2 for requirement of reinforcing steel of Class B in accordance with EN 1992-1-1:2004 Table C.4, shall be replaced by reinforcing steel of Grade 500B in accordance with CS2:2012.

CHAPTER 5 DESIGN OF CONCRETE BRIDGES

5.1 GENERAL

(1) Concrete highway structures and railway bridges shall be designed in accordance with the requirements of BS EN 1992-1-1, BS EN 1992-2, the UK NAs to BS EN 1992-1-1 and BS EN 1992-2, PD 6687-1 and PD 6687-2, unless otherwise specified in this Manual. A detailed list of the relevant documents is included in Appendix A.

(2) Due to the differences in the properties of local aggregate and the properties of locally available cement, the material properties of concrete in Hong Kong differ from the properties given in the Eurocodes and the relevant UK NAs. Appropriate values given in this Chapter shall be followed.

(3) The requirements for materials, products, execution and workmanship given in the General Specification for Civil Engineering Works of the Government of the Hong Kong Special Administrative Region shall be complied with. Reference to European Standards, European Technical Approvals and other standards for construction products and execution of works not specified in the General Specification for Civil Engineering Works, where considered necessary, shall be made only if the provisions therein are appropriate to Hong Kong conditions.

5.2 MATERIAL PROPERTIES OF CONCRETE

5.2.1 Strength

(1) Concrete shall be designated by strength grades which relate to the characteristic cube strength, $f_{ck,cube}$, in accordance with the General Specification for Civil Engineering Works. The corresponding Eurocode strength classes, characteristic cylinder strength, f_{ck}, and other mechanical characteristics necessary for design shall be obtained from Table 5.1. Values given in Table 3.1 of BS EN 1992-1-1 as referred to in Clause 3.1.2(3) of BS EN 1992-1-1 shall not be used.

Table 5.1 – Strength and Short Term Elastic Modulus of Concrete

	Strength classes for concrete											
$f_{ck,cube}$ (MPa)	30	35	40	45	50	55	60	65	70	75	80	85
f_{ck} (MPa)	25	28	32	35	40	45	50	52	56	60	64	70
f_{cm} (MPa)	33	36	40	43	48	53	58	60	64	68	72	78
f_{ctm} (MPa)	1.9	2.1	2.3	2.5	2.7	2.9	3.1	3.1	3.1	3.1	3.1	3.1
$f_{ctk,0.05}$ (MPa)	1.4	1.5	1.6	1.8	1.9	2.0	2.1	2.1	2.1	2.1	2.1	2.1
$f_{ctk,0.95}$ (MPa)	2.5	2.8	3.0	3.3	3.5	3.8	4.0	4.0	4.0	4.0	4.0	4.0
E_{cm} (GPa)	22.2	23.7	25.1	26.4	27.7	28.9	30.0	31.1	32.2	33.2	34.2	35.1

(2) The concrete strength grades to be used shall be limited to a minimum grade of Grade 30 and a maximum grade of Grade 85 which correspond to the C_{min} of C25/30 and C_{max} of C70/85 as given in Table NA.1 of the UK NA to BS EN 1992-2 for Clause 3.1.2(102)P of BS EN 1992-2. The shear strength for concrete of strength grades higher than Grade 60 shall be limited to that of Grade 60 unless there is evidence of satisfactory past performance of the particular mix including the type of aggregates used, and this shall be agreed with the Chief Highway Engineer/Bridges and Structures.

(3) Concrete for the carriageway and superstructures including concrete parapets shall be of Grade 40 or stronger.

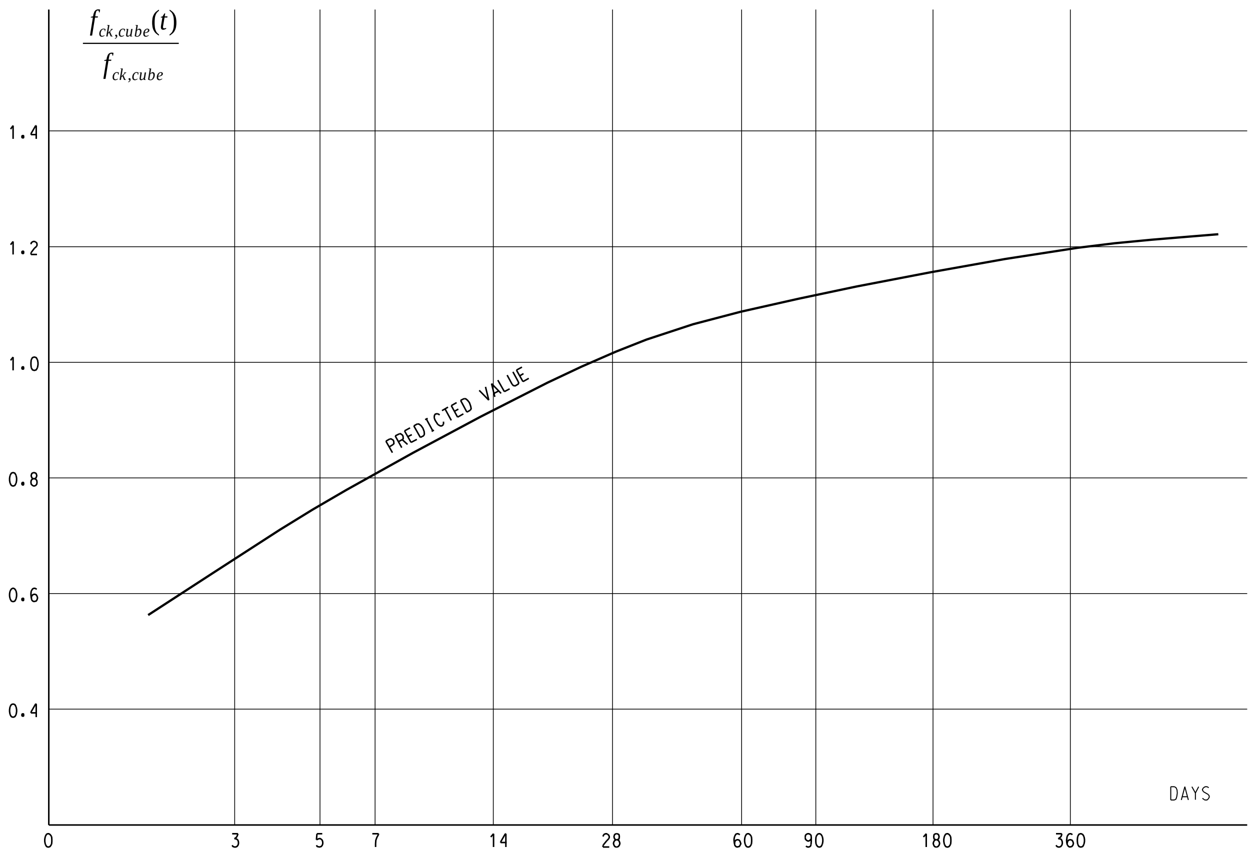
(4) Concrete for prestressing work shall be Grade 45 or stronger. In using concrete of high strength grade (stronger than Grade 60) for prestressing work, strict control shall be exercised with adequate provisions in the contract to ensure the reliability and consistency of the high strength concrete produced.

(5) The provisions given in Clause 3.1.2(5) of BS EN 1992-1-1 for determination of $f_{ck}(t)$, the characteristic compressive cylinder strength of concrete at age t between 3 to 28 days, and in Clause 3.1.2(6) of BS EN 1992-1-1 for estimating $f_{cm}(t)$, the mean compressive cylinder strength at age t, shall not be used. The ratio of characteristic compressive cube strength at age t to that at 28 days, $f_{ck,cube}(t)/f_{ck,cube}$, shall be obtained from Figure 5.1, and $f_{ck}(t)$ shall be determined based on $f_{ck,cube}(t)$ using the relation between $f_{ck}(t)$ and $f_{ck,cube}(t)$ given in Table 5.1. The rate of strength gain indicated in Figure 5.1 applies to OPC concrete only and not to concrete containing PFA or retarding/accelerating admixtures.

(6) For the application of BS EN 1992-1-1 Clause 3.1.2(9), the coefficient for concrete strength at time t, $\beta_{cc}(t)$, shall be taken as the ratio of concrete strengths at time t obtained from Figure 5.1.

(7) For the application of BS EN 1992-2 Clause 6.8.7(101), the coefficient for concrete strength at first load application, $\beta_{cc}(t_o)$, shall be taken as the ratio of concrete strengths at time t_o obtained from Figure 5.1.

Figure 5.1 – Gain of Strength of Concrete



5.2.2 Elastic Deformation

(1) The values for the modulus of elasticity of concrete E_{cm} are significantly lower in Hong Kong than in the UK. The value used for design purposes shall be obtained from Table 5.1. The E_{cm} given in Clause 3.1.3(2) and Table 3.1 of BS EN 1992-1-1 shall not be used.

(2) Variation of the modulus of elasticity with time may be estimated in accordance with Clause 3.1.3(3) of BS EN 1992-1-1 with $f_{cm}(t)/f_{cm}$ to be determined based on $f_{ck,cube}$, $f_{ck,cube}(t)/f_{ck,cube}$ obtained from Figure 5.1 and the relation between f_{cm} and $f_{ck,cube}$ given in Table 5.1.

5.2.3 Linear Coefficient of Thermal Expansion of Concrete

The linear coefficient of thermal expansion shall be taken as 10×10^{-6} per °C for Hong Kong.

5.2.4 Creep

The creep model given in Clause 3.1.4 and Annex B of BS EN 1992-1-1 shall be adopted for both high and normal strength concrete. The creep model for high strength concrete given in Annex B of BS EN 1992-2 shall not be used except Clause B.105 of BS EN 1992-2 for long term delayed strain estimation, which may be used on a project-specific basis.

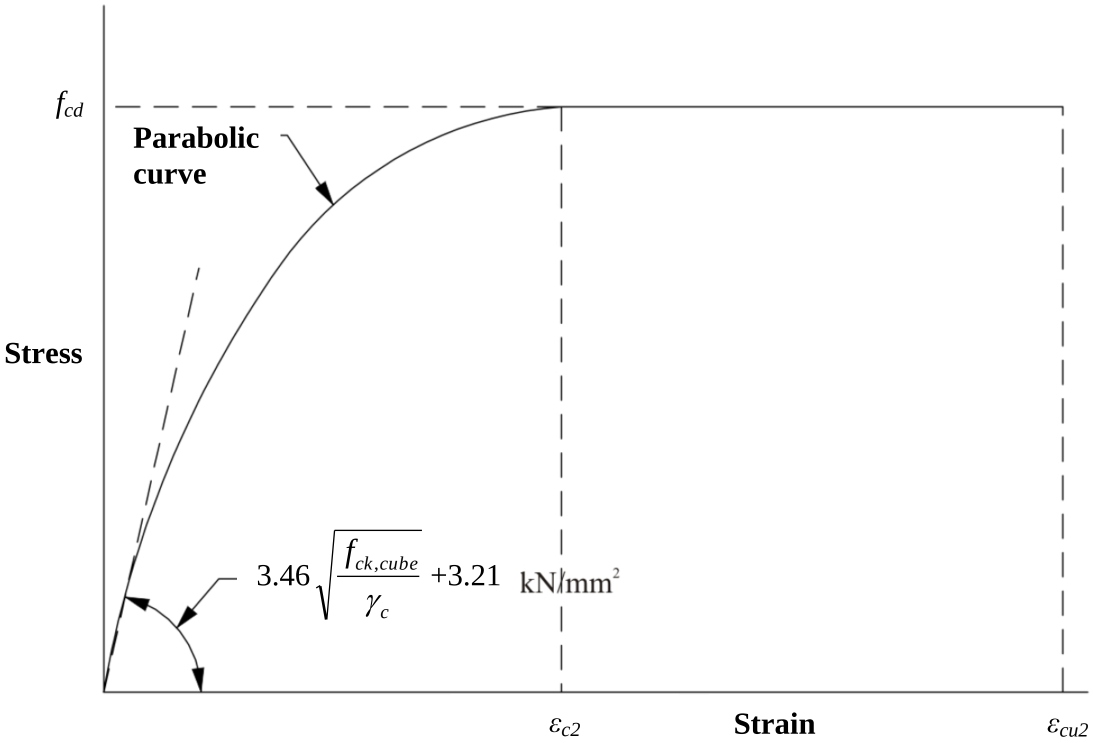
5.2.5 Shrinkage

The shrinkage model given in Clause 3.1.4 and Annex B of BS EN 1992-1-1 shall be adopted for both high and normal strength concrete, except that the values of drying shrinkage strain shall be multiplied by a modification factor of 2.3 to give the drying shrinkage strain for Hong Kong condition. The shrinkage model for high strength concrete given in Annex B of BS EN 1992-2 shall not be used except Clause B105 of BS EN 1992-2 for long term delayed strain estimation, which may be used on a project-specific basis.

5.2.6 Stress-Strain Relations

(1) The stress-strain relation for the design of cross-sections of normal weight and normal weight high-strength concrete is given in Figure 5.2. Alternatively, for the analysis of a cross section to determine its ultimate moment of resistance, the simplified stress block given in Figure 5.3 may be used. These figures are based on the Code of Practice for Structural Use of Concrete 2013 by the Buildings Department. The stress-strain relations given in Figures 3.3 to 3.5 in Clause 3.1.7 of BS EN 1992-1-1 shall not be used.

Figure 5.2 – Design Stress-Strain Curve for Normal Weight Concrete under Compression



NOTE 1 The design compressive strength f_{cd} shall be taken as :

$$f_{cd} = 0.67 \frac{f_{ck,cube}}{\gamma_c}$$

0.67 takes account of the relationship between the cube strength and the bending strength in a flexural member. It is simply a coefficient and not a partial safety factor.

NOTE 2 The strain at reaching the maximum strength ε_{c2} shall be taken as :

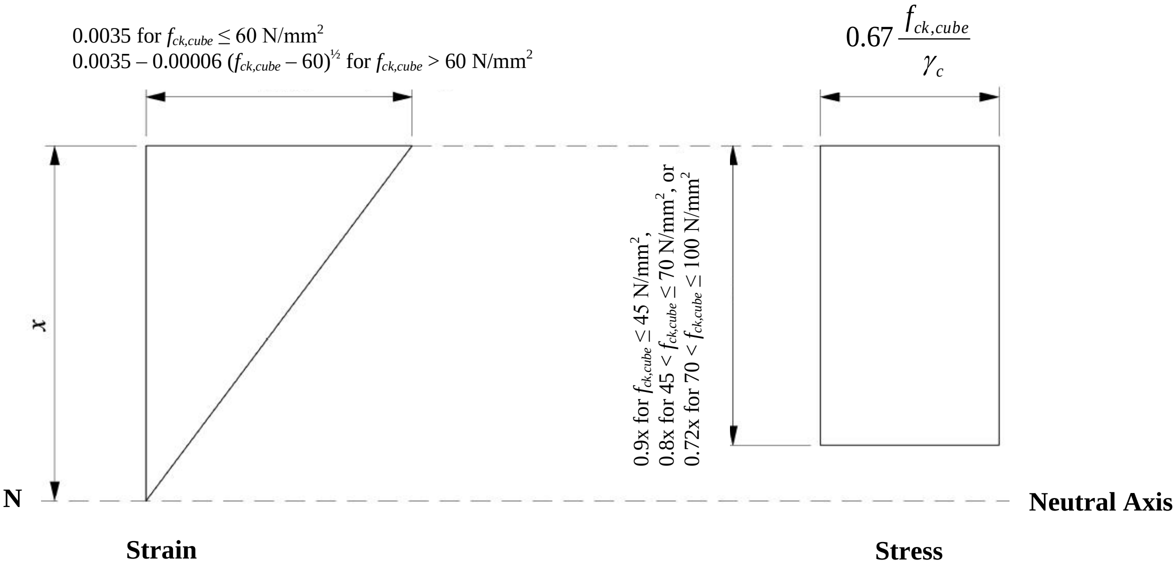
$$\varepsilon_{c2} = \frac{1.34 f_{ck,cube}/\gamma_c}{E_{cm}}$$

NOTE 3 The ultimate strain ε_{cu2} shall be taken as :

$\varepsilon_{cu2} = 0.0035$ for $f_{ck,cube} \leq 60$ MPa
$\varepsilon_{cu2} = 0.0035 - 0.00006 \times \sqrt{(f_{ck,cube} - 60)}$ for $f_{ck,cube} > 60$ MPa

NOTE 4 $f_{ck,cube}$ is in N/mm^2.

Figure 5.3 – Simplified Stress Block for Concrete at Ultimate Limit State



(2) Non-linear structural analysis shall not be used unless with the agreement of the Chief Highway Engineer/Bridges and Structures. Proposed methodology and associated parameters, together with supporting information for their justification, shall be provided for agreement by the Chief Highway Engineer/Bridges and Structures. The stress-strain relation for non-linear structural analysis given in Clause 3.1.5 of BS EN 1992-1-1 shall not be used unless its applicability to Hong Kong is verified by tests.

(3) The increased concrete strength and strain capacities for confined concrete given in Clause 3.1.9(2) of BS EN 1992-1-1 shall not be used in the general design of elements for bending, axial force, shear and torsion. The concrete strength and strain capacities of confined concrete shall be taken as those of unconfined concrete.

5.3 REINFORCING STEEL

(1) Reinforcing steel shall comply with the standards specified in the General Specification for Civil Engineering Works. In particular, hot rolled steel bars shall comply with the requirements of Construction Standard CS2. References in the Eurocodes to reinforcing steel Class B and Class C shall be replaced by grade 500B and grade 500C of CS2:2012 respectively.

(2) Welding of hot rolled high yield steel bars shall not be used.

(3) The requirements and provisions given in Clause 3.2 of BS EN 1991, Clause 3.2.4(101)P of BS EN 1992-2 and the associated clauses in the UK NAs and PD 6687-2 that are in conflict with (1) to (2) above shall not be followed.

5.4 DURABILITY

5.4.1 Cement Content

(1) Cementitious contents shall be in accordance with the requirements of the General Specification for Civil Engineering Works.

(2) For high strength concrete (Grade 60 or above), the total cementitious contents shall be controlled to avoid large heat of hydration as well as large shrinkage and creep strains. Under normal circumstances, the maximum cement content shall be limited to not more than 450 kg/m^3.

5.4.2 Concrete Cover to Reinforcement

The provisions given in Clause 4.4.1 of BS EN 1992-1-1, Clause 4.4.1 of BS EN 1992-2 and the associated clauses of the UK NAs shall be followed except as modified below :

(1) The nominal value of cover for design should be used in the calculations and stated on the drawings.

(2) Nominal concrete covers c_{nom} due to environmental conditions for reinforcing steel and prestressing steel shall be provided in accordance with Table 5.2 for the envisaged conditions of exposure. The provisions given in Clause 4.4.1.2(5) of BS EN 1992-1-1 and the associated UK NA clauses shall not be followed.

(3) Concrete cover of prestressing anchorages shall be provided in accordance with Clause 17.07(2) of the General Specification for General Civil Engineering Works.

(4) Nominal concrete cover c_{nom} for prestressing steel given in Table 5.2 shall be applied also for unbonded tendons.

(5) The provision in Clause 4.4.1.3(3) of BS EN 1992-1-1 on reduction of allowance in design for deviation ΔC_{dev} is not applicable.

Table 5.2 – Nominal Concrete Cover and Crack Width Requirements

Exposure Class	Description of the Environment	Informative examples where exposure classes may occur	Limiting calculated crack width w_{max} (mm)	Nominal Cover c_{nom} [(1)] (mm) Concrete Grade 30	40	45	50	55 & Above
1 No Risk of corrosion or attack								
X0	For concrete without reinforcement or embedded metal: all exposures except where there is abrasion For concrete with reinforcement or embedded metal: very dry	- Concrete in enclosed environments with very low air humidity	0.30	30 (30)	25 (25)	25 (25)	25 (25)	25 (25)
2 Corrosion induced by carbonation								
XC1	Dry or permanently wet	- Concrete in enclosed environments with low air humidity - Concrete permanently submerged in water - Above ground level and fully sheltered against rain e.g. (a) surface protected by water proofing membrane (b) internal surfaces	0.25	45 (45)	40 (40)	35 (40)	35 (40)	35 (40)
XC2	Wet, rarely dry	- Concrete surface subject to long-term water contact		45 (55)	45 (55)	40 (50)	40 (50)	40 (50)
XC3	Moderate humidity	- Concrete in enclosed environments with moderate or high air humidity - External concrete sheltered from rain		-	45 (55)	40(50)	40 (50)	40 (50)
XC4	Cyclic wet and dry	- Concrete surfaces subject to water contact, not within exposure class XC2 - Exposed to driving rain - Subject to alternative wetting and drying e.g. (a) bridge deck soffits (b) buried parts including underside of structures resting on layer of blinding concrete not less than 50 mm thick		-	50 (60)	50 (60)	45 (55)	45 (55)
3 Corrosion induced by chlorides								
XD1	Moderate humidity	- Concrete surfaces exposed to airborne chlorides	0.25	-	55 (65)	55 (65)	50 (60)	50 (60)
XD2	Wet, rarely dry	- Concrete components exposed to industrial waters containing chlorides		-	60 (70)	60 (70)	55 (65)	55 (65)
XD3	Cyclic wet and dry	- Parts of bridges exposed to spray containing chlorides		-	-	65 (75)	65 (75)	60(70)
4 Corrosion induced by chlorides from sea water								
XS1	Exposed to airborne salt but not in direct contact with sea water	- Structures near to or on the coast	0.25	-	55 (65)	55 (65)	50 (60)	50 (60)
XS2	Permanently submerged	- Parts of marine structures permanently submerged in sea water	0.15	-	-	60 (70)	60 (70)	55 (65)
XS3	Tidal, splash and spray zones	- Parts of marine structures directly affected by sea water spray e.g. concrete adjacent to the sea - Exposed to abrasive action by sea water	0.15 0.10	-	-	75 (75)	75 (75)	70 (70)

Notes:
(1) Figures in brackets are nominal covers for prestressing steel.
(2) The nominal covers include an allowance in design for deviation ΔC_{dev} of 10 mm.
(3) For in-situ concrete slab of more than 250 mm thick, the nominal cover derived from this table shall be increased by 5 mm subject to a max of 75 mm.
(4) For underside of structures in contact with ground, the design crack width shall not exceed 0.25 mm while the nominal cover shall be at least 75 mm.
(5) For concrete exposed to water with a pH ≤ 4.5, the grade strength shall not be lower than Grade 45 with a crack width limit of 0.10 mm and a nominal cover of 75 mm.

5.5 CRACK CONTROL

5.5.1 General

(1) Reinforced concrete structures or structural elements shall be designed so that the sum of the calculated crack width under the Crack Width Verification Combination specified in Clause 3.2.2 and the thermal crack width calculated in accordance with Clause 5.5.2, where applicable, does not exceed the limiting calculated crack width w_{max} given in Table 5.2. The recommended values of w_{max} given in Table NA.2 of the UK NA to BS EN 1992-2 shall not be used.

(2) Prestressed concrete structures or structural elements shall be designed so that :

(a) decompression in the tension face of concrete does not occur under the Tensile Stress Verification Combination for Prestressed Concrete Members Case 1 as specified in Clause 3.2.3(1); and

(b) design flexural tensile stresses in the tension face of concrete under the Tensile Stress Verification Combination for Prestressed Concrete Members Case 2 as specified in Clause 3.2.3(2), do not exceed $0.45\sqrt{f_{ck,cube}}$ for pre-tensioned members and $0.36\sqrt{f_{ck,cube}}$ for post-tensioned members as given in Table 5.3.

Table 5.3 – Allowable Flexural Tensile Stress under Tensile Stress Verification Combination for Prestressed Concrete Members Case 2

Type of prestressed member	**Allowable stress (N/mm^2) for concrete grade**			
	35	**40**	**50**	**60 & Above**
Pre-tensioned	-	2.9	3.2	3.5
Post-tensioned	2.1	2.3	2.6	2.8

5.5.2 Early Thermal Movement

(1) Immature concrete expands as a result of the heat released during hydration. Cracking can occur if any part of the immature concrete is restrained from moving when the heat of hydration dissipates, and cooling and contraction take place. Reinforcement shall be provided to control such cracking. Guidance on the calculation of early thermal crack is not given in BS EN 1992-1-1 nor BS EN 1992-2. Provisions given in this Section shall be followed.

(2) The minimum amount of reinforcement to be provided is given by :

$\rho = f_{ct} / f_{yk}$

where ρ = steel ratio

$= A_s / A_c$

f_{ct} = tensile strength of immature concrete

= $(f_{ck,cube}/20)^{2/3}$ approximately

f_{yk} = characteristic yield strength of reinforcement

A_s = cross section area of reinforcement

A_c = gross area of concrete as defined in Clause 5.5.2(3).

(3) The surface of immature concrete cools and contracts before the core. The proportion of reinforcement required shall accordingly be calculated using an "effective surface zone", assumed to be 250 mm thick, on each face for the area of concrete A_c. There are thus two cases to be considered :

(a) members less than 500 mm thick, and

(b) members equal to or more than 500 mm thick.

(4) For members less than 500 mm thick, the steel ratio ρ shall be applied to the whole cross-sectional area to obtain the amount of steel required. This amount shall be provided equally divided between the two faces and shall be provided in each of the two directions.

(5) Considerations of crack widths and spacing generally mean that more reinforcement is required to control cracking than the minimum amount given by the above formula.

(6) The likely maximum spacing of cracks is given by :

$$s = (f_{ct} / f_b) \times (\varphi / 2\rho)$$

where s = maximum crack spacing

f_{ct} / f_b = ratio of tensile strength of immature concrete to average bond strength, which may be taken as

= 1 for plain round bars

= 2/3 for deformed bars

φ = bar size.

(7) For design purposes, the above relation may be more conveniently expressed as:

$$n\varphi \geq (f_{ct} / f_b) \times (2bh / \pi s)$$

where n = number of bars in section

b = width of section

h = depth of member.

(8) The maximum crack width which occurs during cooling from peak hydration temperature to ambient temperature may be taken as :
$w_{max} = s\ a_t\ T_1 / 2$

where a_t = coefficient of thermal contraction of mature concrete

T_1 = fall in temperature between hydration peak and ambient.

(9) The permissible crack width shall be that which is appropriate for the environmental conditions given in Table 5.2 less the crack width resulting from flexure.

(10) The effective thermal coefficient of immature concrete is taken as half the mature value given at Clause 5.2.3.

(11) Any further fall in temperature T_2 due to seasonal variations will also contribute to cracking. Various factors due to ageing seem again to reduce the effect of thermal contraction by about half, so that the combined maximum crack width is :

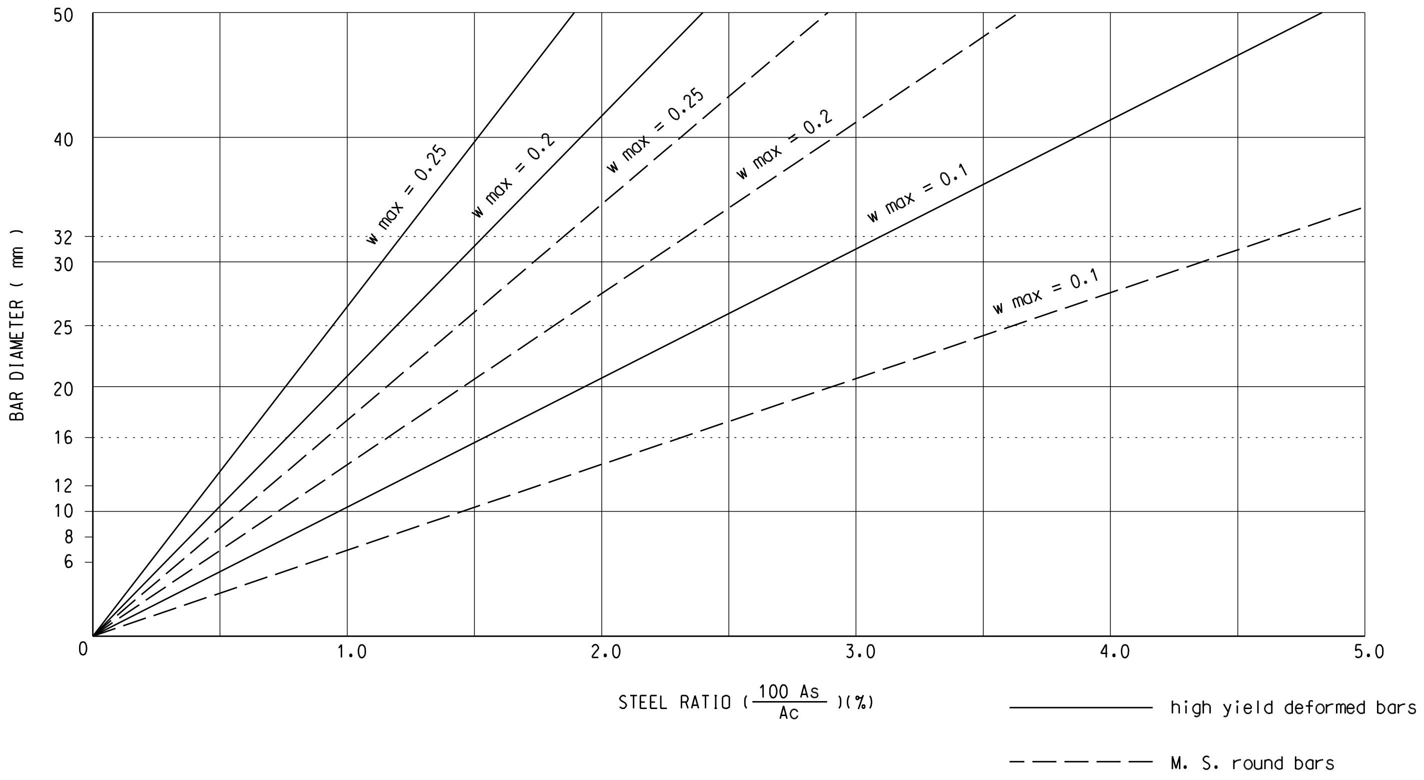
$w_{max} = s\ a_t\ (T_1 + T_2) / 2$

(12) For members less than 15m long, or with movement joints at 15m centres or less, the effect of T_2 may be neglected. T_2 may also be neglected if the restraint is being provided by a section subject to the same climatic exposure as that being restrained.

(13) The formulae given above may be used to determine the amount of reinforcement required to control cracking. Alternatively the amount of reinforcement to be provided may be taken from Figure 5.4, which has been prepared assuming values of 35°C and 30°C respectively for T_1 and T_2 as being representative of Hong Kong conditions.

(14) Reinforcement that is present in the section for other purposes may be included as part of the area of reinforcement necessary to satisfy the requirements for the control of early thermal cracking.

Figure 5.4 – Steel Ratio for Thermal Crack Control



5.6 PRESTRESSING

5.6.1 General

(1) Prestressing steel and devices shall comply with the standards specified in the General Specification for Civil Engineering Works. The requirements and provisions given in Clauses 3.3 and 3.4 of BS 1992-1-1 and the associated clauses in the UK NAs and PD 6687-2 that are in conflict with the requirements of the General Specification for Civil Engineering Works and the clauses in this Section shall not be followed.

(2) For design, stress-strain curve with a horizontal branch as given in Clause 3.3.6(7) of BS EN 1992-1-1 shall be assumed. The assumption of an inclined top branch shall only be used with prestressing steels complying with prEN10138.

5.6.2 Post-tensioning Systems

(1) Various proprietary post-tensioning systems are available in Hong Kong. To avoid any suggestion that the choice of a proprietary post-tensioning system might be influenced by other than engineering considerations, trade names shall not be included in specifications or drawings. Instead, general prestressing requirements shall be given in the contract documents, and the main contractor shall be required to submit detailed proposals to the Engineer for approval showing how one of the acceptable proprietary post-tensioning systems may be used to apply the required prestressing forces.

(2) Such general requirements may include, as appropriate, any or all of the followings :

(a) number, location and profile of prestressing tendons;

(b) number of wires, strands or bars per tendon;

(c) size and type of wire, strand or bar (standard, high-strength, compacted; normal or low relaxation);

(d) anchorage type (dead-end, coupling or stressing-end);

(e) sequence of stressing the tendons;

(f) prestressing force; and

(g) ducting and grouting requirements.

(3) The contract documents shall make clear whether the value of prestressing force includes losses due to :

(a) relaxation of steel;

(b) elastic deformation of concrete;

(c) shrinkage and creep of concrete;

(d) friction and wobble;

(e) draw-in,

where appropriate giving details of any assumption made, and also making clear whether allowance shall be made for anchorage and jack losses.

(4) Consideration must be given at the design stage to the practicability of fitting one or other of the acceptable proprietary post-tensioning systems into the work being designed, so that the post-tensioning specialists are not set an impossible task. End-block reinforcement depends on the type of anchorage used, and so shall not be detailed, but, again, consideration shall be given at the design stage to likely requirements. The proposals submitted by the main contractor must accordingly include end-block reinforcement details.

5.6.3 External Prestressing

(1) All highway structures and railway bridges adopting external prestressing shall be checked to ensure that failure of any two external tendons or 25% of the tendons at any one section, whichever is the greater, will not lead to collapse at ultimate limit state under the design values of permanent actions.

(2) All external tendons shall be replaceable and provisions shall be made within the design for inspection, removal and replacement of any external tendon.

(3) A robust multiple barrier protection system shall be used to prevent the external tendons from weathering and corrosion.

5.6.4 Specialist Prestressing Contractors

(1) All prestressed concrete works for highway structures shall be carried out by specialist contractors in the Prestressed Concrete Works for Highway Structures Category of the List of Approved Suppliers of Materials and Specialist Contractors for Public Works.

(2) The Prestressed Concrete Works for Highway Structures Category consists of two classes:

Class I - Supply and Installation of Prestressing Systems; and

Class II - Supply of Prestressed Concrete Units.

(3) The supply and installation of on-site prestressing work shall be carried out by a contractor in Class I. Precast prestressed units manufactured off-site shall be supplied by a contractor in Class II.

5.7 ADDITIONAL MODIFICATIONS TO BS EN 1992 AND THE UK NA TO BS EN 1992

In addition to those indicated in the preceding sections, the modifications to BS EN 1992-1-1, BS EN 1992-2, the UK NAs to BS EN 1992-1-1 and BS EN 1992-2, PD 6687-1 and PD 6687-2 as given in Table 5.4 shall be followed.

Table 5.4 – Additional Modifications to BS EN 1992-1-1, BS EN 1992-2, the UK NAs to BS EN 1992-1-1 and BS EN 1992-2, PD 6687-1 and PD 6687-2

Item	Clause of Eurocodes	Contents	Modifications
5.7.1	BS EN 1992-1-1 Clause 2.4.2.4(1)	Partial factors for materials for ultimate limit states	The partial factors γ_C and γ_S given in Table 2.1N of BS EN 1992-1-1 for persistent & transient design situations shall also be used for accidental design situations.
5.7.2	BS EN 1992-1-1 Clause 2.4.2.4(3)	Lower values of partial factors for materials	Lower partial factors shall only be used with the agreement of the Chief Highway Engineer/Bridges and Structures.
5.7.3	BS EN 1992-1-1 Clause 2.5	Design assisted by testing	Design assisted by testing shall only be used with the agreement of the Chief Highway Engineer/Bridges and Structures.
5.7.4	BS EN 1992-1-1 Clause 2.6	Supplementary requirements for foundations	This clause is not applicable. Provisions given in Chapter 10 of this Manual shall be followed.
5.7.5	BS EN 1992-1-1 Clause 5.6.2	Plastic analysis for beams, frames and slabs	Plastic analysis shall only be used for the check at ULS and only with the agreement of the Chief Highway Engineer/Bridges and Structures. Sufficient ductility shall be demonstrated to the satisfaction of the Chief Highway Engineer/Bridges and Structures.

Item	Clause of Eurocodes	Contents	Modifications
5.7.6	BS EN 1992-1-1 Clauses 5.10.2.1 and 5.10.3(2)	Maximum stressing force for prestressing tendons and limitation of initial prestress force	Immediately after anchoring, the force in the prestressing tendon shall not exceed 70% of the characteristic strength for post-tensioned tendons, or 75% for pre-tensioned tendons. The jacking force may be increased to 80% during stressing provided that additional consideration is given to safety, to the stress-strain characteristics of the tendon, and to the assessment of the friction losses. Provisions given in BS EN 1992-1-1 Clauses 5.10.2.1 and 5.10.3(2) shall only be followed when prestressing tendons comply with prEN10138.
5.7.7	BS EN 1992-1-1 Clause 5.10.2.2(5)	Limitation of concrete compressive stress at transfer	The values of k_6 shall be taken as 0.6.
5.7.8	BS EN 1992-1-1 Clauses 6.2.3(2) and 6.2.4(4)	Limiting values of angles θ for the shear truss model	The angle between the concrete compression strut and the beam axis perpendicular to the shear force θ, shall be limited to 45°.
5.7.9	BS EN 1992-2 Clause 6.3.2(102)	Design Procedure for torsion – Limiting value of strut inclination θ	The angle of strut inclination shall be limited to 45°.
5.7.10	BS EN 1992-1-1 Clause 7.2(5)	Stress limit to avoid appearance of unacceptable cracking or deformation	The value of k5 shall be taken as 0.7 unless prestressing tendons comply with prEN10138.
5.7.11	BS EN 1992-1-1 Clause 7.3.3(2)	Control of cracking without direct calculation	The simplified method of crack control without direction calculation given in Clause 7.3.3(2) of BS EN 1992-1-1 shall not be used.

Item	Clause of Eurocodes	Contents	Modifications
5.7.12	BS EN 1992-2 Clause 8.10.3	Anchorage zones of post-tensioned members	Reference shall be made to CIRIA guide 1. Requirements in Annex J.104.2 of BS EN 1992-2 shall also be checked and adopted where more onerous than CIRIA guide 1.
5.7.13	BS EN 1992-1-1 Clause 9.2.1.2(3)	Rules on confinement of compression longitudinal reinforcement	The diameter of transverse reinforcement shall comply with the requirements in Clause 9.5.3(101) of BS EN 1992-2.
5.7.14	BS EN 1992-1-1 Clause 9.5.2(4)	Minimum area of longitudinal reinforcement in columns	The number of longitudinal bars in a circular section shall not be less than six.
5.7.15	BS EN 1992-1-1 Clause 9.6.2	Minimum area and spacing of vertical reinforcement in walls	Requirements on minimum reinforcement given in Clause 9.5.2(2) of BS EN 1992-1-1 shall apply. The distance between two adjacent vertical bars shall not exceed 3 times the wall thickness or 300 mm whichever is the lesser.
5.7.16	BS EN 1992-1-1 Section 11	Lightweight aggregate concrete structures	Lightweight aggregate concrete shall only be used with the agreement of the Chief Highway Engineer/Bridges and Structures. Proposed material parameter to suit local Hong Kong conditions and methodology for fatigue verification, together with supporting information for their justification, shall be provided for agreement by the Chief Highway Engineer/Bridges and Structures.
5.7.17	BS EN 1992-1-1 Annex A	Modification of partial factors for material	This annex shall only be used with the agreement of the Chief Highway Engineer/Bridges and Structures.
5.7.18	BS EN 1992-1-1 Annex C	Properties of reinforcement suitable for use with Eurocode	This annex shall be for reference only.

CHAPTER 6 DESIGN OF STEEL BRIDGES

6.1 GENERAL

(1) Steel highway structures and railway bridges shall be designed in accordance with the requirements of BS EN 1993-1, BS EN 1993-2, the UK NAs to BS EN 1993-1 and BS EN 1993-2, PD6695-1-9, PD6695-1-10 and PD 6695-2, unless otherwise specified in this Manual. A detailed list of the relevant documents is included in Appendix A.

(2) The requirements for materials, products, execution and workmanship given in the General Specification for Civil Engineering Works of the Government of the Hong Kong Special Administrative Region shall be complied with. Reference to European Standards, European Technical Approvals and other standards for construction products and execution of works not specified in the General Specification for Civil Engineering Works, where considered necessary, shall be made only if the provisions therein are appropriate to Hong Kong conditions.

(3) Material and workmanship for structural steelwork shall comply with BS 5400: Part 6 in so far as its requirements are appropriate to Hong Kong conditions unless specified otherwise in the General Specification for Civil Engineering Works. Where the requirements or conditions stipulated in BS 5400: Part 6 differ from Hong Kong conditions, adjustments appropriate to Hong Kong shall be made as necessary.

6.2 FATIGUE

(1) Special attention shall be given to fatigue assessment during the design of bridges which are particularly prone to fatigue and fracture damage, such as cable-stayed bridges or steel bridges that are frequently used by heavy vehicles.

(2) The provisions given in BS EN 1993-1-9, the UK NA to BS EN 1993-1-9 and PD 6695-1-9 for fatigue assessment shall be followed in so far as they are appropriate to Hong Kong conditions. Where the provisions therein differ from Hong Kong conditions, adjustments appropriate to Hong Kong shall be made as necessary.

6.3 HOT FORMED STRUCTURAL HOLLOW SECTIONS

(1) Hot formed hollow sections with steel properties and section sizes in accordance with BS EN 10210 Part 1 and Part 2 respectively shall be used for all structural steelworks.

(2) Designers shall check that the sections proposed will be available in the quantities required before finalising the design. The use of cold formed sections as an alternative shall not be permitted.

6.4 FABRICATION

(1) Structural steelwork shall be fabricated and erected by specialist contractors in the "Structural Steelwork" category of the List of Approved Suppliers of Materials and Specialist Contractors for Public Works.

(2) All structural steelworks shall be detailed so that they can be hot dip galvanized after fabrication, and also, they can be erected without damaging the galvanizing and without on site welding. For long span trusses and structures too large for hot dip galvanizing after fabrication, consideration shall be given to the application of sprayed metal coating after fabrication. If non-ferrous components are used with steel fixings, insulation must be provided to prevent galvanic corrosion.

(3) Hot rolled steel sections shall be blast cleaned and protected with blast primers before fabrication and welding. This prevents the development of rust, which would be difficult to remove after fabrication. The use of steel that has rusted heavily during storage shall not be allowed for the same reason.

(4) When welding metal coated or zinc dust painted steel, the coating near the weld area shall first be removed, or the weld area be masked off before coating. After welding, scale and heat damaged coatings shall be removed by local blast cleaning and the areas renovated by re-applying the original coating. Damaged galvanized or metal sprayed surfaces shall be made good by :

(a) metal spraying;

(b) application of zinc rich paints to reinstate the original dry film thickness; or

(c) application of low melting point zinc alloy heated by torch to a pasty condition with the fluxes contained therein removed.

(5) The aforesaid guidelines shall not be applicable to exceptionally massive steelwork, such as the steel deck of the Tsing Ma Bridge, Ting Kau Bridge, etc., where special corrosive protection system shall be considered with regard to the particular project requirements.

6.5 BLAST CLEANING

Blast cleaning of steelworks shall be carried out by specialist contractors in the "Class V: Hot dip galvanizing" of the "Specialized Operations for Highway Structures" category of the List of Approved Suppliers of Materials and Specialist Contractors for Public Works.

6.6 TESTING OF WELDS

(1) All structurally important welds of structural steelwork shall be subject to nondestructive testing in the form of magnetic particle, liquid penetrant, radiographic or ultrasonic inspection and interpretation by specialist contractors in the "Class IV: Non-destructive testing of welds" of the "Specialized Operations for Highway Structures"

category of the List of Approved Suppliers of Materials and Specialist Contractors for Public Works. The extent of testing shall comply with the requirements given in BS 5400: Part 6, Clause 5.5.2. The designer shall specify the welds to be tested above this requirement.

(2) Destructive testing of welds for steelwork shall be carried out in accordance with BS 5400: Part 6, Clauses 5.5.1.1, 5.5.1.2 and 5.5.1.3.

6.7 HOT DIP GALVANIZING

All hot dip galvanized steel components shall comply with BS EN ISO 1461 after fabrication by specialist contractors in the "Class V: Hot dip galvanizing" of the "Specialized Operations for Highway Structures" category of the List of Approved Suppliers of Materials and Specialist Contractors for Public Works. Steel hollow sections shall be sealed wherever this can be done without affecting the galvanizing process. If venting is necessary, the vents shall be carefully detailed and positioned so as to be inconspicuous, or be effectively sealed immediately after galvanizing.

6.8 ADDITIONAL MODIFICATIONS TO BS EN 1993 AND THE UK NA TO BS EN 1993

In addition to those indicated in the preceding sections, the modifications to BS EN 1993-1, BS EN 1993-2, the UK NAs to BS EN 1993-1 and BS EN 1993-2 and PD 6695-2 as given in Table 6.1, shall be followed.

Table 6.1 – Additional Modifications to BS EN 1993-1, BS EN 1993-2, the UK NAs to BS EN 1993-1 and BS EN 1993-2 and PD 6695-2

Item	Clause of Eurocodes	Contents	Modifications
6.8.1	BS EN 1993-1-1 Clause 5.3.2	Imperfections for global analysis of frames	Second order analysis shall only be used with the agreement of the Chief Highway Engineer/Bridges and Structures.
6.8.2	BS EN 1993-1-1 Clause 5.3.3	Imperfection of bracing system	Second order analysis shall only be used with the agreement of the Chief Highway Engineer/Bridges and Structures. In the absence of second order analysis, Clauses 10 and 11 of PD 6695-2 shall be followed for the design of restraints at supports and intermediate restraints respectively.

Item	Clause of Eurocodes	Contents	Modifications
6.8.3	BS EN 1993-1-1 Clause 5.3.4	Member imperfections	Second order analysis shall only be used with the agreement of the Chief Highway Engineer/Bridges and Structures.
6.8.4	BS EN 1993-2 Clause 6.1(1)P	Partial factors	The partial factors γ_{Mi} given in Clause NA.2.17 of the UK NA to BS EN 1993-2 shall be used except for the following: γ_{M0} = 1.05 $\gamma_{M6,ser}$ = 1.05 γ_{M3} = 1.3 $\gamma_{M3,ser}$ = 1.2
6.8.5	BS EN 1993-1-1 Clause 6.2.2.4(1)	Effective properties of cross section with class 3 web and class 1 or 2 flanges	This clause shall not be used.
6.8.6	BS EN 1993-1-1 Clause 6.3.1.2	Buckling curves	Buckling curve a_0 given in Figure 6.4 of BS EN 1993-1-1 shall not be used and shall be replaced by curve a.
6.8.7	BS EN 1993-1-1 Clause 6.3.2.2	Lateral torsional buckling curves for general case	Calculation of the elastic critical moment for lateral-torsional buckling M_{cr} by finite element model shall only be used with the agreement of the Chief Highway Engineer/Bridges and Structures. In the absence of such analysis, either the method given in Clause 6.3.4.2(2) of BS EN 1993-2 (where applicable) or one of those given in Clauses 5 to 8 of PD 6695-2 as appropriate, shall be followed.
6.8.8	BS EN 1993-2 Clause 6.5	Buckling of plates	For plates with out of plane loading, Clause 12 of PD 6695-2 shall be followed.
6.8.9	BS EN 1993-1-8 Clause 3.6.1	Bolts and rivets	Values of k_2 for tension resistance given in Table 3.4 of BS EN 1993-1-8 shall be replaced by the following : k_2 = 0.63 for countersunk bolt, otherwise k_2 = 0.7

Item	Clause of Eurocodes	Contents	Modifications
6.8.10	BS EN 1993-1-8 Clause 4.3.5(1)	Plug welds	Plug welds shall not be used to transmit load.
6.8.11	BS EN 1993-1-8 Clause 4.8	Design resistance of plug welds	Plug welds shall not be used to transmit load.
6.8.12	BS EN 1993-2 Annex A	Technical specifications for bearings	The provisions in this annex may be followed unless specified otherwise in Chapter 8 of this Manual.
6.8.13	BS EN 1993-2 Annex B	Technical specifications for expansion joints for road bridges	This annex shall not be used. Provisions given in Chapter 9 of this Manual shall be followed.
6.8.14	BS EN 1993-2 Annex C Clause C3.2	Tolerances for fabrication	Structural details 3), 4) and 5) for deck to stiffener connection in Table C.4 of BS EN 1993-2 shall be modified as follows: (1) To minimize the risk of fatigue damage to the stiffener-deck plate welds, in addition to the maximum 2 mm lack of penetration to the back of the stiffener, the target penetration shall be 90% ± 5% of the stiffener thickness. (2) The maximum fit-up gap between stiffener and deck shall be 0.25 mm over lengths of 200 mm.
6.8.15	BS EN 1993-1-5 Clause 2.5	Non uniform members	Finite element methods shall only be used with the agreement with the Chief Highway Engineer/Bridges and Structures.
6.8.16	BS EN 1993-1-5 Clause 2.6	Members with corrugated webs	Member with corrugated webs shall only be used with the agreement with the Chief Highway Engineer/Bridges and Structures.
6.8.17	BS EN 1993-1-5 Clauses 6.1 to 6.6	Resistance to transverse forces	The partial factor γ_{M1} shall be taken as 1.3 for Clause 6.1 to 6.6 of BS EN 1993-1-5.

Item	Clause of Eurocodes	Contents	Modifications
6.8.18	BS EN 1993-1-5 Clause 7.1	Interaction between shear force, bending moment and axial force	For longitudinally stiffened girders, the design plastic moment of resistance of the cross section $M_{pl,Rd}$ shall be taken as the same as the design elastic moment of resistance of the cross section $M_{el,Rd}$.
6.8.19	BS EN 1993-1-5 Clause 9.2.1(9)	Minimum requirements for transverse stiffeners	The parameter θ shall be taken as 6.0.
6.8.20	BS EN 1993-1-5 Clause 10(5)	Reduced stress method	The method given in Clause 10(5)(a) of BS EN 1993-1-5 shall be not used. For Clause 10(5)(b) of BS EN 1993-1-5, Expression (10.5) shall be replaced with: $\left(\frac{\sigma_{x,Ed}}{\rho_x f_y/\gamma_{M1}}\right)^2+\left(\frac{\sigma_{z,Ed}}{\rho_z f_y/\gamma_{M1}}\right)^2-V\left(\frac{\sigma_{x,Ed}}{\rho_x f_y/\gamma_{M1}}\right)\left(\frac{\sigma_{z,Ed}}{\rho_z f_y/\gamma_{M1}}\right)+3\left(\frac{\tau_{Ed}}{\chi_w f_y/\gamma_{M1}}\right)^2\leq 1.0$ where $V = \rho_x \rho_z$ when σ_{xEd} and σ_{zEd} are both compressive $V = 1.0$ otherwise
6.8.21	BS EN 1993-1-5 Clause B.1(3)	Non-uniform members	The reduction factors shall be obtained from Sections 4 and 5 of BS EN 1993-1-5. Expressions (B.1) and (B.2) given in Clause B.1(3)NOTE of BS EN 1993-1-5 shall not be used.
6.8.22	BS EN 1993-1-5 Annex C	Finite element methods of analysis for plated structures	This annex shall only be used with the agreement of the Chief Highway Engineer/Bridges and Structures.
6.8.23	BS EN 1993-1-5 Annex D	Plate girders with corrugated webs	Members with corrugated webs shall only be used with the agreement of the Chief Highway Engineer/Bridges and Structures.
6.8.24	BS EN 1993-1-9 Clause 3	Assessment Methods	Damage tolerant method shall not be used for fatigue assessment.

CHAPTER 7 DESIGN OF COMPOSITE BRIDGES

7.1 GENERAL

(1) Composite highway structures and railway bridges shall be designed in accordance with the requirements of BS EN 1994-1-1, BS EN 1994-2, the UK NAs to BS EN 1994-1-1 and BS EN 1994-2, and PD 6696-2, unless otherwise specified in this Manual. A detailed list of the relevant documents is included in Appendix A.

(2) The requirements for materials, products, execution and workmanship given in the General Specification for Civil Engineering Works of the Government of the Hong Kong Special Administrative Region shall be complied with. Reference to European Standards, European Technical Approvals and other standards for construction products and execution of works not specified in the General Specification for Civil Engineering Works, where considered necessary, shall be made only if the provisions therein are appropriate to Hong Kong conditions.

(3) Modifications to BS EN 1992, BS EN 1993 and the relevant UK NAs given in Chapter 5 and Chapter 6 of this Manual, where applicable to the design of composite highway structures and railway bridges, shall be followed unless otherwise specified in this Chapter.

7.2 ADDITIONAL MODIFICATIONS TO BS EN 1994 AND THE UK NA TO BS EN 1994

In addition to those indicated in the preceding section, the modifications to BS EN 1994-1-1, BS EN 1994-2, the UK NAs to BS EN 1994-1-1 and BS EN 1994-2, and PD6696-2 as given in Table 7.1, shall be followed.

Table 7.1 – Additional Modifications to BS EN 1994-1-1, BS EN 1994-2, the UK NAs to BS EN 1994-1-1 and BS EN 1994-2, and PD6696-2

Item	Clause of Eurocodes	Contents	Modifications
7.2.1	BS EN 1994-2 Clause 5.4.3(1)P	Non-linear global analysis for bridges	Non-linear analysis shall only be used with the agreement of the Chief Highway Engineer/Bridges and Structures.
7.2.2	BS EN 1994-2 Clause 5.4.4	Combination of global and local effects	Provisions given in Clause 3.4 of PD 6696-2 shall also be followed.
7.2.3	BS EN 1994-2 Clause 6.2.2	Resistance to vertical shear	The contribution of the attached concrete slab shall be ignored in the determination of the shear resistance.

Item	Clause of Eurocodes	Contents	Modifications
7.2.4	BS EN 1994-2 Clause 6.4	Lateral-torsional buckling of composite beams	Provisions applicable to the elastic critical moment for lateral-torsional buckling M_{cr} given in Chapter 6 of this Manual shall also be followed. Clause 4.6 of PD 6696-2 shall also be referred to for guidance.
7.2.5	BS EN 1994-2 Clause 6.6.3.2	Headed stud connectors in solid slabs and concrete encasement – Influence of tension on shear resistance	Provisions given in Clause 5.1 of PD 6696-2 shall also be followed.
7.2.6	BS EN 1994-2 Clause 6.7.2	General method of design	Non-linear analysis shall only be used with the agreement of the Chief Highway Engineer/Bridges and Structures.
7.2.7	BS EN 1994-2 Clause 6.8.1(3)	Fatigue – Longitudinal shear force for headed stud shear connectors in bridges	The factor k_s shall be taken as 0.675.

CHAPTER 8 BEARINGS

8.1 GENERAL

(1) Highway structures and railway bridges flex, expand and contract. Bearings shall be provided at appropriate locations to enable such movements to take place freely and without damage to the structures. They shall be positioned to minimize the out of balance forces.

(2) The design and installation of bearings shall follow the recommendations of BS 5400 : Part 9 : Section 9.1 : Code of Practice for Design of Bridge Bearings and BS 5400 : Part 9 : Section 9.2 : Specification for Materials, Manufacture and Installation of Bridge Bearings respectively in so far as these recommendations are appropriate to Hong Kong conditions. Alternatively, the recommendations of other national standards may be followed subject to the prior approval of the Chief Highway Engineer/Bridges and Structures.

(3) Bridge bearings shall not be subjected to uplift forces under any combinations of actions unless with the prior approval of Chief Highway Engineer/Bridges and Structures.

8.2 CLASSIFICATION OF BEARINGS

(1) Many proprietary brands of bearing are available commercially. However, trade names for proprietary bearings shall not be included in specifications or drawings to avoid any suggestion that the choice might be influenced by other than engineering considerations.

(2) Bearing requirements shall be given in general terms, using the classification given in Table 8.1 as an aid to specifying.

8.3 SCHEDULE OF BEARINGS

(1) A schedule of bearings shall be prepared for all contracts covering highway structures and railway bridges for which bearings are required. Such a schedule shall detail the number and performance requirements for each class of bearing required for the contract. Concurrent vertical and horizontal loads shall be defined in the bridge bearing schedule. A specimen schedule is given in Table 8.2

(2) If the maximum transverse and longitudinal loads are considered as acting with maximum vertical load, overdesign is likely in the majority of cases and the design is therefore not economical. Therefore, the designers shall specify different load combinations in the bridge bearing schedule.

Table 8.1 – Classification of Bearings

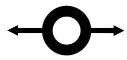
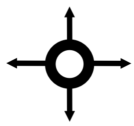
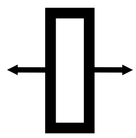
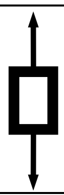
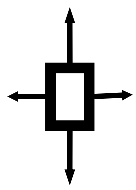
Rotation Class	Translation Class		Bearing Class
rotation all round	no translation		point rocker
			pot
			fixed elastomeric
			spherical
			compound cylindrical
	translation in one direction		constrained point rocker sliding
			constrained pot sliding
			constrained elastomeric
			constrained spherical sliding
			constrained compound cylindrical
	translation in all directions		free point rocker sliding
			free pot sliding
			free elastomeric
			free spherical sliding
			free compound cylindrical
rotation about one axis only	no translation		line rocker
			cylindrical
			fixed elastomeric
			pot
			spherical
	translation perpendicular to rotational axis		roller
			constrained line rocker sliding
			constrained cylindrical sliding
			constrained pot sliding
			constrained spherical sliding
	translation parallel to rotational axis		cylindrical sliding
			constrained line rocker sliding
			constrained pot sliding
			constrained spherical sliding
	translation in all directions		constrained roller sliding
			free rocker sliding
			free cylindrical sliding
			free pot sliding
			free spherical sliding
	translation in one direction		guide - no vertical load
no rotation	translation in one direction		guide - no vertical load

Table 8.2 – Bridge Bearing Schedule

Bridge name or reference						
Bearing identification mark						
Number of bearing						
Classification of bearing	Rotation Class					
	Translation Class					
	Bearing Class					
Seating material	Upper surface					
	Lower surface					
Allowable average contact pressure (N/mm²)	Upper face	Serviceability				
		Ultimate				
	Lower face	Serviceability				
		Ultimate				
Design load effects (kN)	Serviceability limit state		Vertical	Maximum		
				Permanent		
				Minimum		
			Transverse			
			Longitudinal			
	Ultimate limit state		Vertical			
			Transverse			
			Longitudinal			
Translation (mm)	Serviceability limit state	Irreversible	Transverse			
			Longitudinal			
		Reversible	Transverse			
			Longitudinal			
	Ultimate limit state	Irreversible	Transverse			
			Longitudinal			
		Reversible	Transverse			
			Longitudinal			
Rotation (radians)	Serviceability limit state	Irreversible	Transverse			
			Longitudinal			
		Reversible	Transverse			
			Longitudinal			
	Maximum rate (100 x radians / kN)		Transverse			
			Longitudinal			
Maximum bearing dimensions (mm)	Upper surface		Transverse			
			Longitudinal			
	Lower surface		Transverse			
			Longitudinal			
	Overall height					
Tolerable movement of bearing under transient loads (mm)			Vertical			
			Transverse			
			Longitudinal			
Allowable resistance to translation under serviceability limit state (kN)			Transverse			
			Longitudinal			
Allowable resistance to rotation under serviceability limit state (kN.m)			Transverse			
			Longitudinal			
Type of fixing required			Upper face			
			Lower face			

8.4 SUPPLY AND INSTALLATION OF BEARINGS

The bearings shall be supplied and installed by specialist contractors in the "Supply and Installation of Bearings for Highway Structures" category of the List of Approved Suppliers of Materials and Specialist Contractors for Public Works. As such only those proprietary bearings already approved for supply and installation by these specialist contractors shall be used.

8.5 TESTING

(1) The scope of testing and the test loads shall be specified in the Specification. The number and type of bearing tests must also be clearly stated in the Specification and itemized in the Bills of Quantities.

(2) Bearings designed and manufactured in accordance with the provisions of BS 5400 : Part 9 : Sections 9.1 and 9.2 will not normally require ultimate limit state testing.

8.6 COMPRESSIVE STIFFNESS OF ELASTOMERIC LAMINATED BEARINGS

(1) BS 5400 : Part 9 : Section 9.1 adopts the expression "$5GS^2$" to evaluate the compressive stiffness of elastomeric laminated bearings. Apparently, the width to length ratio of the rectangular bearing is not taken into consideration in this expression. It has been pointed out in the "Malaysian Rubber Products Association, 1981, Code of Practice" (MRPA) that the coefficient of "5" is only sufficiently correct for a "long thin bearing with a width to length ratio of 0.25 but is more accurate for a ratio of 0.20 or less" (Reference - P.B. Lindley, Small-strain compression and rotation moduli of bonded rubber blocks, Plastics and Rubber Processing and Applications 1 (1981)).

(2) Since the width to length ratio and the compressive stiffness curve is not linear for rectangular bearings, the "MRPA" has recommended "CGS^2" instead of "$5GS^2$", and

$$C = 4 + \frac{b_e}{l_e}\left(6 - 3.3\frac{b_e}{l_e}\right)$$

$$\left.\begin{array}{lll} \text{where} \quad b_e & = & \text{effective bearing width} \\ \quad\quad\quad\; l_e & = & \text{effective bearing length} \end{array}\right\} \; b_e < l_e$$

(3) It has been found that test results generally agree with the calculated values by using this "C" value, whereas the BS 5400 : Part 9 : Section 9.1 prediction is usually too soft. Hence the MRPA's recommendation shall be followed.

8.7 DESIGN OF FIXINGS FOR BRIDGE BEARINGS

(1) Except for elastomeric bearings, bridge bearings, including bearings which are not required to provide horizontal restraint, shall be fixed to the superstructure and substructure with mechanical fixings fabricated from austenitic stainless steel. The grade of the stainless steel shall comply with the followings :

Wrought stainless steel	:	Grade 1.4436 to BS EN 10250-4
Flat rolled stainless steel	:	Grade 1.4436 to BS EN 10088
Stainless steel washers	:	Grade 1.4436 to BS EN 10088
Stainless steel fasteners	:	Grade A4-80 to BS EN ISO 3506-1 and BS EN ISO 3506-2

(2) The friction between the bearing and the superstructure or substructure may be used to resist part of the horizontal forces provided that a factor of safety of at least 2 is applied to the proven coefficient of friction and that the worst combination of vertical load and horizontal load is applied.

(3) For seismic design situation, friction shall only be considered if the vertical reaction can be reasonably predicted.

8.8 OPERATIONAL REQUIREMENTS

(1) The possibility that bearings may need to be replaced during the design lifetime of a bridge must be recognized. Provision shall therefore be made in the design for the removal and replacement of bearings without causing any undue damage to the structures. The jacking force and the jacking position for bearing replacement shall be indicated on the drawings. Where special procedures need to be followed for the replacement of bearings, a method statement shall be indicated on the drawings. Alternatively, such a statement shall be submitted to the maintenance authority at the time of handover of the completed structure.

(2) Where access to bearings would otherwise be difficult or impossible, special arrangements shall be included in the design to enable access to be obtained. In particular, bearings at halving joints would render the inspection and maintenance to be difficult or impossible. Hence, halving joints shall be used only in exceptional circumstances with the prior approval of the maintenance authority and only where adequate access for inspection and maintenance is provided. Sufficient space shall be provided for bearings to be properly inspected and maintained. In case of doubt, the maintenance authority shall be consulted.

(3) Bearings shall be detailed so that dirt and rubbish do not accumulate around them, and that they can easily be cleaned. They shall be detailed so that moisture cannot stand in their vicinity but will instead drain away elsewhere. In this connection, reference shall be made to Clause 9.5.

CHAPTER 9 MOVEMENT JOINTS

9.1 GENERAL

9.1.1 Movements

(1) Highway structures undergo dimensional changes as a result of temperature changes, shrinkage, creep and the application of the prestress. Live loads cause bearings to deflect and rotate, and bearings can also produce movements if carriageways are inclined. The resulting movements, which are illustrated in Figure 9.1 , shall be determined at the design stage, and provision shall be made for such movements to take place without damage to the structures.

(2) Movement joints shall accordingly be included in the design of highway structures to accommodate anticipated movements. However, as movement joints are very difficult to repair or replace once the structures are opened to traffic, no more than the minimum number shall be provided, even if a continuous or otherwise redundant structure has to be chosen for this reason.

(3) Transverse movement of the decks of curved or skew bridges can occur, causing damage to movement joints. Joints which can accommodate the transverse movement shall be used. Otherwise, guided bearings shall be provided to prevent any transverse movement.

(4) Longitudinal movement joints shall not be used unless unavoidable since such joints adversely affect the riding comfort and safety of vehicular traffic.

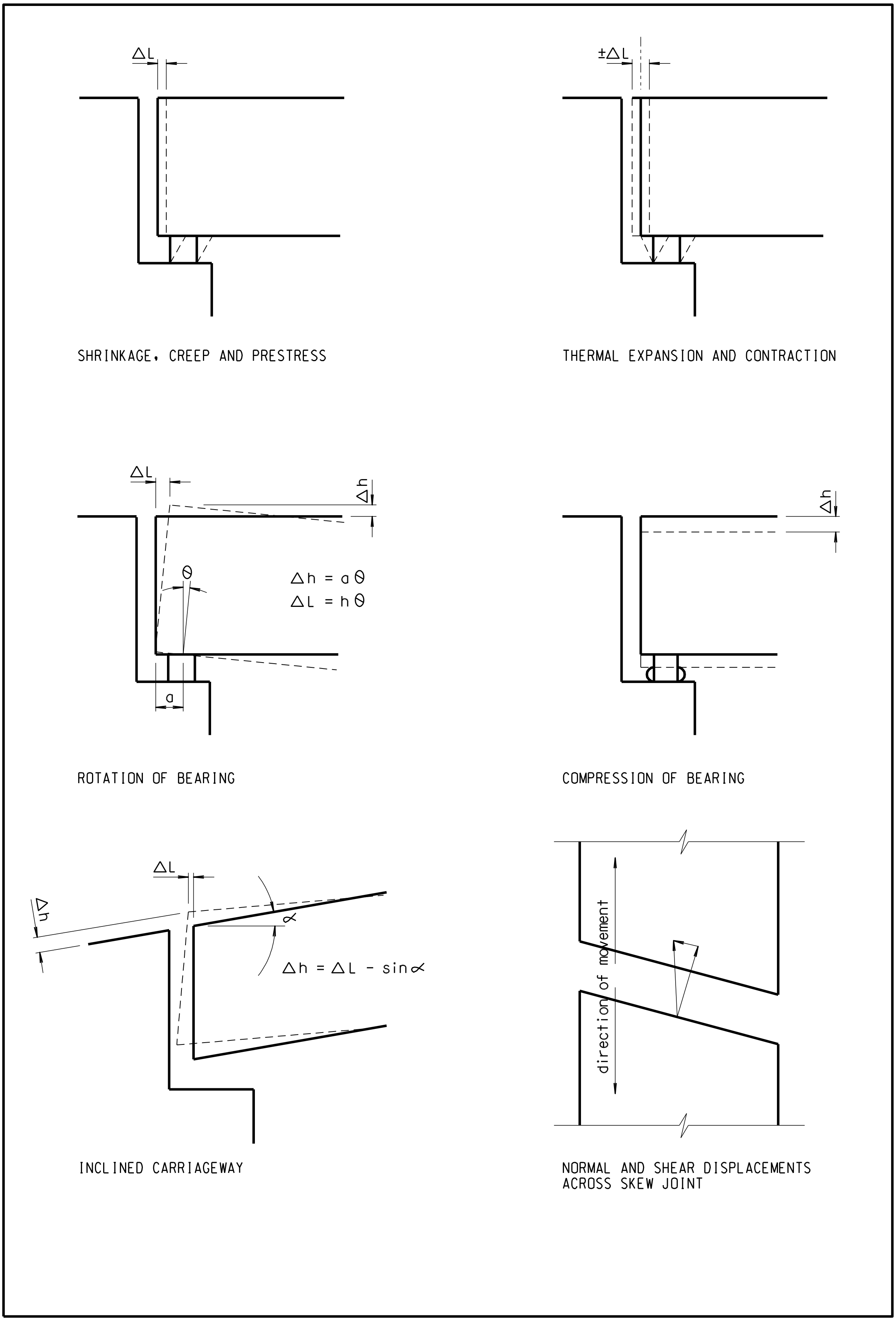
9.1.2 Selection of Joint Type

(1) Selection of the type of joint to be provided is mainly determined by the total movement to be expected.

(2) Where movements up to ±5 mm are expected, gap, filled or buried joints shall normally be used. Gap joints are, of course, the simplest and easiest, but are only suitable for vehicular traffic. They shall not be used where pedestrians or cycles are expected.

(3) Filled joints may consist of a gap filled with a compressible filler and sealed with a suitable sealant, or the top of the gap may be filled with a proprietary rubber or neoprene sealing strip inserted under compression and supported by rebates. Careful detailing of such joints is necessary to ensure that the filler does not fall out, leaving the sealant unsupported, when the joint opens; or that the compressive forces holding the sealing strip in place do not decrease when the joint opens enough for the sealing strip to jump out. The maximum gap width for such joints is 25 mm. Filled joints transmit a proportion of a horizontal force in the deck on one side of the joint to the adjacent deck, and this shall be borne in mind at the design stage.

Figure 9.1 – Joint Movement



(4) Buried joints may be used on structures with asphaltic surfacing. In such joints, the asphaltic surfacing is separated from the gap by means of cover plates or other devices designed to spread out the movement over the length of the plates or devices across the joints, which shall be fabricated from corrosion resistant material, or a special flexible surfacing is used over the gap. Such joints must be designed and constructed with great care, as cracks tend to reflect up from the gap, causing the surfacing to deteriorate rapidly. The maximum gap width shall not exceed 20 mm. This kind of joint is not suitable for bridge decks sloping more than 1 in 30.

(5) Proprietary movement joints shall be used on all structures that carry vehicular traffic.

(6) The maximum size of continuous open gap which can be tolerated for motor vehicles is 65 mm. Where pedestrians and cyclists have access, all gaps shall be sealed and covered with non-slip cover plates fabricated from corrosion resistant material. If the gap is sealed with anything other than a hard, load bearing rubber, then so far as the riding quality is concerned, the joint shall be considered as an open gap.

9.2 PROPRIETARY MOVEMENT JOINTS

(1) A large variety of proprietary movement joints is now available. Care shall be taken to ensure that :

(a) a movement joint inherently suitable for the required location is chosen;

(b) the design of the structure is capable of accommodating the movement joint selected (there will be no conflict between joint fixings and prestress anchorages/bursting steel or steel reinforcement etc.); and

(c) the installation is carried out so that the properties of the selected movement joint are fully exploited.

(2) Experience has shown that correct installation of proprietary movement joints is essential for satisfactory performance.

(3) Overestimation of shrinkage and creep movements can result in the joint being constantly under compression and bowing upwards after installation thereby generating excessive noise during the passage of vehicular traffic. The movement to be expected shall accordingly be estimated with the greatest possible accuracy; in this particular application, over-estimation is not on the safe side. A slight downward tilt of the mountings, so that the joint sags under compression rather than hogs, may reduce this particular problem.

9.3 TRAFFIC LOADING ON MOVEMENT JOINTS

(1) Movement joints shall be able to carry the same vehicular loads as the structures of which they are parts. For structures designed to carry the vehicular loads described in this Manual, movement joints and their holding down bolts shall be capable of withstanding the following loads, either separately or in combination :

(a) vertically : two 112.5 kN wheel loads, 1000 mm apart, each spread over a contact area giving an average pressure of 1 N/mm², applied so as to give the worst possible effect;

(b) horizontally : a traction force of 75 kN per linear metre of movement joint acting at road level, combined with any forces that may result from straining the joint filler or seal.

(2) The minimum diameter of holding down bolts shall be 16 mm. Holding down bolts and the component parts shall be fabricated from austenitic stainless steel.

(3) For prestressed holding down and fixing arrangements, the size of bolts could be reduced provided they have sufficient elastic working capacity.

9.4 LOADING OF STRUCTURE BY STRAINING OF MOVEMENT JOINTS

(1) As movement joints open and close under the influence of temperature changes, shrinkage, creep and loadings, the proprietary components of such joints may be strained, depending on their design, and forces may be transmitted to the supporting structures.

(2) Allowance shall be made at the design stage of the structures for such forces. The force that a joint may exert on the supporting structure shall not be more than 5 kN/m, but for design purposes of the supporting structures a value of at least 20 kN/m shall be assumed.

9.5 WATERTIGHTNESS OF MOVEMENT JOINTS

(1) Unsealed movement joints enable stormwater to penetrate onto the bearings, piers and abutments of highway structures and railway bridges. Such penetration is undesirable as it can cause corrosion of ferrous bearing components, staining of exposed surfaces and produces an undesirable appearance.

(2) Stormwater penetration through movement joints may be dealt with in three ways :

(a) a proprietary movement joint designed so that the completed installation is watertight may be chosen (although in practice such joints are always liable to leakage and some means of drainage shall accordingly always be provided);

(b) a proprietary movement joint which allows the passage of stormwater may be used in conjunction with a drainage layer or channel added to catch stormwater and divert it to the drainage system; or

(c) the structure may be designed so that stormwater can pass freely through the movement joint to be collected on the piers and abutments and diverted to the drainage system without accumulating around bearings or staining exposed surfaces but such drainage system must be capable of being easily inspected and maintained. This is the most reliable method for structure with large movement joints.

A conscious decision shall be made at the design stage as to which of these alternatives is to be followed.

(3) Stormwater draining through track ballast onto a railway underbridge must be collected and led away. Not only shall joints be carefully sealed, but in addition a substantial heavy duty waterproofing membrane shall be applied to the bridge deck. The waterproofing membrane shall be continued across the deck ends and taken down behind the ballast walls, with drains to collect and remove water running down the membrane. The membrane shall be suitably protected against damage by track ballast.

9.6 FUNCTIONAL REQUIREMENTS OF PROPRIETARY MOVEMENT JOINTS

9.6.1 Requirements

Proprietary movement joints selected for use on highway structures and railway bridges shall satisfy the following requirements :

(a) it shall withstand traffic loads and accommodate movements of the bridge and shall not give rise to unacceptable stresses in the joint or other parts of the structure;

(b) it shall be easy to inspect and maintain, and parts liable to wear shall be easily replaceable;

(c) large metal surfaces exposed at road level shall have skid resistant surface treatments;

(d) it shall have good riding quality and shall not cause inconvenience to any road user (including cyclists and pedestrians where they have access);

(e) the joint shall not generate excessive noise or vibration during the passage of traffic;

(f) it shall either be sealed or have provision for carrying away water, silt and grit;

(g) joints with exposed rubber running surfaces shall not be used for new vehicular bridges;

(h) the holding down and fixing arrangements for the joints shall be effectively concealed at the carriageway level.

9.6.2 Specification

(1) In addition to specifying the requirements listed in Clause 9.6.1 above, the specifications for movement joints shall include :

(a) the maximum allowable gap opening;

(b) the limiting force that the joint may exert on the structure;

(c) the minimum size of holding down bolt to be provided; and

(d) a schedule of movement joint detailing the number and performance requirements to be provided. A specimen schedule is given in Table 9.1.

(2) Trade names for proprietary movement joints shall not be included in specifications and drawings to avoid any suggestion that the choice might be influenced by other than engineering considerations.

9.7 SUPPLY AND INSTALLATION OF MOVEMENT JOINTS

(1) The movement joints shall be supplied and installed by specialist contractors in the "Supply and Installation of Expansion Joints for Highway Structures" category of the List of Approved Suppliers of Materials and Specialist Contractors for Public Works. As such only those proprietary movement joints already approved for supply and installation by these specialist contractors shall be used.

(2) Other proprietary movement joints complying with the specifications and drawings in all respects may also be acceptable. However the specialist firm dealing with the supply and installation of these movement joints must first be included for such in the "Supply and Installation of Expansion Joints for Highway Structures" category of the List of Approved Suppliers of Materials and Specialist Contractors for Public Works.

(3) The main contractor engaged on projects involving the supply and the installation of movement joints shall be required to submit to the Engineer for approval full details of the proprietary movement joints he proposes to use. The details provided shall fully describe and illustrate the proposed method of installation of the movement joints.

(4) The Chief Highway Engineer/Bridges and Structures shall be consulted in case of doubt about the performance or suitability of any particular brand/type of movement joint.

9.8 DETAILING FOR PROPER INSTALLATION OF MOVEMENT JOINTS

(1) Installation of proprietary movement joints consists mainly of the following operations :

(a) preparation of seating;

(b) installation of holding down systems;

(c) provision of bedding;

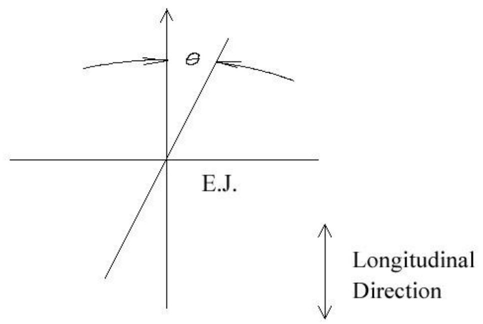
(d) fixing of joint;

(e) provision of nosing and backing.

Table 9.1 – Schedule of Movement Joint

Ref. No.	Qty. (no.)	Length (mm)	Longitudinal Movement (after 180 days of concreting)			Transverse Movement (mm)	Direction of Movement (Θ)
			Irreversible (mm)	Reversible (mm)			
				due to LL	thermal		

Notes: (1) For Longitudinal movement "+ve" denotes closing of joints
"-ve" denotes opening of joints



(2) Reversible thermal movements shall be calculated based on the followings :

Coefficient of thermal expansion 10×10^{-6} / °C for concrete
12×10^{-6} / °C for structural steel

Temperature range Refers to Table 3.17 and Table 3.18

Mean temperature at setting 20ºC

(3) The contractors shall make adjustments when the setting temperature is different from 20°C. Such adjustment shall be subject to the approval of the Engineer.

(4) Irreversible longitudinal movements shall be calculated assuming the joints were to be installed no sooner than 180 days after the last pour of concrete for the deck structure. If the contractor wishes to install the joint earlier, he shall adjust the anticipated movements. Such adjustment shall be subject to the approval of the Engineer.

(2) Execution of most of these operations can be made more effective if careful consideration is given at the design stage to accessibility, working space and so on. Movement joints are usually installed at the ends of concrete members, where simplicity of formwork produces better results than more complex arrangements. Hooks and bends in reinforcement tend to be concentrated at such points, more from lack of thought by the designers than necessity. The possibility of bond and anchorage consideration permitting reinforcement to be stopped off well away shall be investigated, because many movement joint failures have arisen through holding down bolts not being properly accommodated among unnecessarily congested underlying reinforcement. The joints between the recesses in concrete decks for elastomeric movement joints and the concrete in the adjacent concrete decks shall be formed by saw cutting to a depth of 25 mm. The surface of fabricated movement joints shall be at least 1 mm, and not more than 3 mm, below the surrounding road surface.

9.9 OPERATIONAL REQUIREMENTS

Where modular type movement joints are proposed, provision shall be made for access to the underside of the joints for inspection and repair / replacement of parts liable to wear and tear. In case of doubt, the maintenance authority shall be consulted.

CHAPTER 10 FOUNDATIONS AND SUBSTRUCTURES

10.1 FOUNDATIONS

10.1.1 General

(1) Foundations of highway structures shall be designed in accordance with the principles set out in BS 8004, as well as the provisions given in the latest editions of Geotechnical Manual for Slopes and Geoguides published by the Geotechnical Engineering Office, Civil Engineering and Development Department. Pile design shall make reference to GEO Publication No. 1/2006 - "Foundation Design and Construction".

(2) BS 8004 has not been drafted on the basis of limit state design, but it will be appropriate to adopt the design actions specified in Chapter 3 and Chapter 4 of this Manual with γ_G , γ_P and γ_Q taken as 1 for the purpose of verifying foundations in accordance with BS 8004.

(3) Design situations and combinations of actions to be considered in the design of foundations shall be in accordance with BS EN 1990 and the corresponding UK NA as supplemented by this Manual.

(4) For pile design, global factors of safety as stated in GEO Publication No. 1/2006 shall be applied.

(5) Structural design of piles and pile caps shall be in accordance with the limit state philosophy to Eurocodes as supplemented by this Manual, unless otherwise provided for in GEO Publication No. 1/2006.

(6) Earth pressure on abutment walls shall be determined in accordance with the latest edition of Geoguide 1 - "Guide to Retaining Wall Design" together with amendments published by the Geotechnical Engineering Office, Civil Engineering and Development Department.

10.1.2 Specialist Piling Contractors

(1) Piling works shall be carried out by specialist contractors in the "Land Piling" category of the List of Approved Suppliers of Materials and Specialist Contractors for Public Works.

(2) Some piling contractors use specialized methods or designs, with features particularly suited to certain sites. To take advantage of such features, apart from giving details of the conforming piling designs in the tender documents, details of the forces and moments to be resisted by the piles shall also be included to allow for submission of alternative designs. Tenderers submitting alternative designs shall be asked to price for both the conforming and alternative designs so that any financial benefits are easily recognisable.

10.1.3 Piling Downdrag

In Hong Kong, where reclamations often overlay compressible marine mud, the phenomenon of "downdrag" or "negative skin friction" may occur, when the weight of the surrounding soil is transferred to the piles as consolidation takes place. Extra loads on piles arising from this effect shall be assessed in accordance with Clauses 6.8 and 7.4 of GEO Publication No. 1/2006.

10.1.4 Differential Settlement

(1) Differential settlement shall be taken as a permanent load, which shall be assessed separately for each highway structure support taking into account the foundation type, loading intensity and subsoil conditions.

(2) The elastic modulus of concrete used in conjunction with the differential settlement effect shall be the long term elastic modulus.

10.1.5 Cover to Pile Caps and Footings

A desirable minimum cover of 1.5 m should be provided for pile caps and footings of highway structures to facilitate the installation of future utilities.

10.2 BRIDGE SUBSTRUCTURES ADJACENT TO RAIL TRAFFIC

(1) Recommendations concerning accidental actions caused by derailed trains colliding with the bridge substructures are given in Clause 3.6.3 of this Manual.

(2) The best defence against impact from derailed trains is to site the supports of railway overbridges well away from the railway track, preferably at least 5 m from the centre line of the nearest track. If space limitations make remote siting of supports impossible, the following precautions shall be observed :

   (a) Supports shall not be pin-jointed at both top and bottom.

   (b) A solid plinth shall be provided around individual columns to a height of 1000 mm above adjacent rail level, with "cut-water" shaped ends to deflect derailed trains.

   (c) In the absence of solid plinth, the bottom part of the support shall be of "cut-water" shape to deflect derailed trains.

   (d) In case a support is formed by a group of individual columns, the support shall be designed such that the loss of any one column in that group will not lead to failure of the support under the combinations of actions for accidental design situation after an accident event.

(3) To ensure reasonable robustness, supports shall be designed to withstand, without collapse, a minimum static design force of 1650 kN in case of vehicular bridges, and 825 kN in case of footbridges, acting horizontally in any direction at a height of 1200 mm above the adjacent rail level in conjunction with permanent actions and other variable actions under the accidental design situation.

10.3 RAILWAY UNDERBRIDGE SUBSTRUCTURES

Railway underbridges shall be provided with ballast walls at approaches, high enough and long enough to prevent ballast from falling on to abutments and wing walls. The ends of wing walls will normally adjoin the boundary of the railway, where they shall be at least 2 m high above the adjoining pavement level.

10.4 HYDRAULIC EFFECTS ON BRIDGE SUBSTRUCTURES

10.4.1 Effects to be Considered

(1) Consideration shall be given to the effects of :

(a) pressure due to currents;

(b) hydrostatic pressure;

(c) scour;

(d) backwater; and

(e) waterborne traffic.

(2) Assessment of the above effects shall make reference to the provisions given in BA 59/94 - "The Design of Highway Bridges for Hydraulic Action" of the United Kingdom Highways Agency, as far as they are applicable to Hong Kong conditions.

(3) If a structure is exposed to the sea, the effects of wave action shall also be considered. Reference could be made to "Port Works Design Manual" (PWDM) published by the Civil Engineering and Development Department in assessing the effects of wave action. In addition, the recommended specification given in Appendix B of the PWDM Part 1 for reinforced concrete in marine environment should be adopted to address the corrosion concern.

10.4.2 Backwater Effects

(1) For a bridge crossing a river or stream, consideration shall be given to the backwater effects produced by the highway or railway crossing restricting the flow of water. Backwater can cause flooding upstream of the crossing and, in addition, the increased velocity of the stream, and its turbulence, can cause scour sufficient to endanger the bridge structure.

(2) If piers or supporting structures are to be located within drainage channels or natural watercourse, Drainage Services Department shall be consulted to see whether a Drainage Impact Assessment is required.

10.4.3 Effects of Waterborne Traffic

(1) The design of piers for bridges over navigation channels shall include consideration of protection against ship collision.

(2) In general, such protection is costly, and the risk involved shall be carefully analysed and weighed against the possibility of protecting the lives of bridge users by means such as those described in Clause 16.2 of this Manual.

10.5 RUN-ON-SLABS

(1) Run-on-slabs shall not be provided where bituminous carriageway is adopted. It is considered more economical to rely on proper compaction of backfill behind abutments and to make up the carriageway surface as settlement occurs.

(2) In the case of a concrete carriageway, the carriageway slab adjacent to the abutment may be designed as a run-on-slab supported off the ballast wall of the abutment. However, it should be noted that proper compaction of the fill behind the abutment is vital for the satisfactory performance of the run-on/carriageway slab without creating maintenance problems in service.

CHAPTER 11 PARAPETS

11.1 GENERAL

(1) A parapet is a structural component installed along the edge of a bridge or similar structure. Parapets are basically of three categories :

(a) vehicle parapets, designed to contain vehicles only on a structure;

(b) pedestrian parapets, designed to safeguard pedestrians but not to contain vehicles; and

(c) bicycle parapets, designed to safeguard cyclists but not to contain vehicles.

(2) Besides containing vehicles and safeguarding pedestrians and cyclists on a structure, parapets may have other purposes such as :

(a) to shield something below from view;

(b) to reduce noise pollution; and

(c) to prevent splash, from stormwater, or other missiles reaching the area below.

(3) In order to minimize maintenance problems arising from the proliferation of parapet designs, parapets shall as far as possible be of the standard designs having due regard to the appearance and functions of the structure. The outer, non-traffic, profile of standard concrete vehicle parapets may however be altered to suit the bridge architecture.

(4) The Chief Highway Engineer/Bridges and Structures shall be consulted at an early stage in the design of the structure for advice on the updated list of standard parapet designs. If special considerations suggest that the use of standard parapet designs appears to be inappropriate for any reason in a particular structure, the prior agreement of the relevant maintenance authorities and the Chief Highway Engineer/Bridges and Structures must be obtained for adopting non-standard designs.

11.2 VEHICLE PARAPET GROUPS

11.2.1 Containment Levels

(1) The range of possible vehicular impacts onto a parapet is extremely large in terms of vehicle type, approach angle, speed and other road conditions. For standardisation, the performance of a parapet is defined in terms of containment level based on a standardised impact configuration.

(2) Vehicle parapets are classified into four groups of performance classes of containment levels as given in Table 11.1. Vehicle characteristics are given inTable 11.2.

11.2.2 Selection Guidance

(1) Guidance on the selection of containment level are given in Table 11.3. The scoring system referred to for containment level L3 is detailed in Table 11.4.

(2) Designers shall exercise judgment to consider the use of higher containment parapets where accidents risks are very high and the consequences of accidents are serious.

11.3 PARAPET HEIGHTS

(1) Height of parapet shall not be less than the dimensions given in Table 11.5. Height shall be measured from the adjoining paved surface to the top of the parapet. The "adjoining paved surface" is the paved area on the traffic side of a parapet, adjacent to the plinth or base of a parapet.

(2) Parapets higher than the dimensions given in Table 11.5 shall be provided wherever special circumstances require a greater height, in which case designers should note that extra working width may need to be allowed to cater for parapet deformation and vehicle movement during accident.

11.4 DESIGN DETAILS

11.4.1 Materials

Parapets may be constructed of steel, aluminium alloy, reinforced concrete or combinations of these materials.

11.4.2 Projections and Depressions

(1) Vehicle parapet shall have traffic face free of projections and depressions, except at joints in longitudinal members but all such projections or depressions shall not exceed 20 mm.

(2) Longitudinal rails shall be placed on the traffic side of their supporting posts, and present a smooth face to traffic free from sharp edges. The front faces of the longitudinal rails shall be in the plane of the traffic faces and, in no case, may depart from it by more than 25 mm.

11.4.3 Structures Not Exclusively Used as Vehicular Bridges

(1) For structures not exclusively used as vehicular bridges, vehicle parapets shall be positioned adjacent to the carriageway on the structure with pedestrian or bicycle parapets at the back of the footways or cycle tracks as appropriate.

(2) If space is limited and the traffic flow is light and slow, the vehicle parapets may be installed along the edges of the structure with the prior agreement of the Chief Highway

Engineer/Bridges and Structures. In such case, only reinforced concrete vehicle parapets with minimum 800 mm high concrete plinth and metal top rail(s) minimum 1100 mm high above the adjoining paved surface may be used. The reinforced concrete vehicle parapets shall not be set back farther than 3500 mm from the edge of the carriageway in order to avoid the possibility of high angle impacts developing, the consequence of which can be particularly serious.

11.5 METAL PARAPETS AND TOP RAILS

11.5.1 Design Requirements

(1) Subject to the containment level and other requirements in this Manual, metal vehicle parapets shall be designed and fabricated in accordance with the requirements of BS 6779 Part 1 - Specification for Vehicle Containment Parapets of Metal Construction in so far as its recommendations are appropriate to Hong Kong conditions.

(2) Similarly, combined metal and concrete vehicle parapet shall be designed in accordance with the requirements of BS 6779 Part 3 - Specification for Vehicle Containment Parapets of Combined Metal and Concrete Construction in so far as its recommendations are appropriate to Hong Kong conditions.

(3) A vehicle parapet shall be demonstrated to achieve the required containment level with a full-scale impact test or a method agreed by the Chief Highway Engineer/Bridges and Structures. Impact tests and acceptance criteria shall follow BS EN 1317 – Road Restraint Systems, except the vehicle occupant impact severity assessment indices, and other requirements in this Manual.

(4) Metal pedestrian parapets shall be designed and fabricated in accordance with the requirements of BS 7818 - Specification for Pedestrian Restraint Systems in Metal in so far as its recommendations are appropriate to Hong Kong conditions.

(5) Metal bicycle parapets shall be designed and fabricated in accordance with the same requirements of metal pedestrian parapets in so far as they are applicable to metal bicycle parapets.

(6) Where Hong Kong specifications or conditions differ from the requirements or conditions described in the British Standards, adjustments appropriate to Hong Kong shall be made.

(7) The holding down and fixing arrangements of the parapets shall be fabricated from austenitic stainless steel and be of the base plate mounting type. Stainless steel, except those for fasteners, shall be Grade 1.4436 instead of Grade 1.4401 specified in the General Specifications for Civil Engineering Works.

11.5.2 Corrosion

(1) Steel parapets and top rails shall be detailed so that they can be hot dip galvanized properly after fabrication, and so that they can be erected without damaging the

galvanizing and without on site welding. Special attention shall be given to details at joints to prevent water being trapped there.

(2) All steel components shall be hot dip galvanized in accordance with BS EN ISO 1461 to a minimum average mass coating of 600 g/m^2 after fabrication. Accidentally damaged galvanizing shall be made good by :

(a) metal spraying;

(b) application of zinc rich paints to reinstate the original dry film thickness; or

(c) application of low melting point zinc alloy heated by torch to a pasty condition with the fluxes contained therein removed.

(3) Steel hollow sections shall be sealed wherever this can be done without affecting the galvanizing process. If venting is necessary, the vents shall be carefully detailed and positioned so as to be inconspicuous, or be effectively sealed immediately after galvanizing.

(4) Non-ferrous components, particularly of aluminium alloy, do not normally corrode. If non-ferrous components are used with steel fixings, additional protective measures such as insulation must be provided to prevent bimetallic corrosion.

(5) To ensure a reasonable resistance to corrosion, the minimum section thickness of metal members for pedestrian and bicycle parapets shall be :

	Thickness
sealed steel hollow sections	4 mm
unsealed steel sections	5 mm
non-ferrous sections	3 mm

11.5.3 Plinth

(1) A reinforced concrete plinth, whose height at the traffic face (See Figure 11.1) shall be at least 50 mm but not more than 100 mm higher than the adjoining paved surface at any point on the cross section, shall always be provided under a metal parapet where the main structure is of concrete. The plinth shall be sufficiently strong to withstand moments and shears developed at post fixings.

(2) The bottom edge of a plinth shall lie in the plane of the traffic face. The front face shall be in this plane but may be inclined at up to 1 in 12 away from the traffic face up to a maximum of 25 mm. The top of the plinth shall fall toward the traffic face to avoid staining the outside face of the structure. The plinth shall be effectively sealed at the movement joints to prevent water leakage.

11.5.4 Bedding

(1) The bedding used between the base plates of the parapets and the plinth, or between the base plates of the top rails and the concrete parapet shall be capable of permanently transmitting the loads involved, safely and without undue deformation.

(2) The finished bedding shall not contain voids and shall be resistant to penetration by water. It shall have a minimum thickness of 10 mm and a maximum thickness of 30 mm plus allowance for falls on the top of the plinth or concrete parapet. The edge of the bedding shall be not less than 20 mm from the edge of the chamfer or corner of the outside face of the plinth or concrete parapet.

11.6 REINFORCED CONCRETE PARAPETS

11.6.1 Design Requirements

(1) Design requirements for reinforced concrete parapets are given below and in Table 11.6.

(2) The parapet shall include end sections extending 3 m from the ends of the parapet, or on each side of an unconnected vertical joint, and intermediate sections extending between the end sections. The top of the parapet shall as far as possible fall toward the traffic face to avoid staining the outside face of the structure.

(3) The parapet shall be effectively sealed at the movement joints to prevent water leakage. The cover plates assembly over the movement joints on the traffic faces shall be fabricated from austenitic stainless steel with a minimum 2.5% molybdenum composition. Stainless steel, except those for fasteners, shall be Grade 1.4436 instead of Grade 1.4401 specified in the General Specification for Civil Engineering Works.

(4) Concrete shall be of Grade 40 or stronger depending on conditions of exposure. Distribution steel amounting to 50% of the main reinforcement shall be provided in both traffic and outer faces.

(5) The minimum ultimate moment of resistance against vertical bending at base for end sections shall be 33% greater than the values for intermediate sections given in the Table 11.6.

11.6.2 Longitudinal Effects

Reinforced concrete parapets shall normally be designed from considerations of transverse resistance only. A parapet shall not be considered as a longitudinal structural member for stiffening the edge of the structure. Joints shall accordingly be provided to prevent longitudinal structural action but the spacing should generally not be less than 3 m.

11.7 PEDESTRIAN PARAPETS

(1) Pedestrian parapets shall be designed to resist the loading given in Table 11.8. The limiting dimensions for pedestrian parapets are given in Table 11.9 and Figure 11.4.

(2) There shall be no footholds or projections on the traffic face where pedestrians have access. The infilling shall normally be of vertical bars spanning between effective longitudinal members but may be inclined at an angle of not more than 45° to the vertical.

(3) Pedestrian parapets shall be capable of withstanding design loading equivalent to Class 3 pedestrian restraint system of BS 7818, i.e. a uniformly distributed load of 1.4 kN/m when applied at the upper longitudinal member separately in the horizontal and vertical directions.

(4) Prior approval from the relevant maintenance authorities and the Chief Highway Engineer/Bridges and Structures shall be obtained for adopting special design (such as design loading for other specified classes of BS 7818) or any deviation from the requirements mentioned.

11.8 BICYCLE PARAPETS

(1) Bicycle parapets shall be designed to resist the loading given in Table 11.8. They shall be capable of withstanding a uniformly distributed load of 1.4 kN/m when applied at the upper longitudinal member separately in the horizontal and vertical directions. The limiting dimensions for bicycle parapets are given in Table 11.10 and Figure 11.5.

(2) There shall be no footholds or projections on the traffic face where pedestrians have access. The infilling shall normally be of vertical bars spanning between effective longitudinal members but may be inclined at an angle of not more than 45° to the vertical.

(3) Prior approval from the relevant maintenance authorities and the Chief Highway Engineer/Bridges and Structures shall be obtained for adopting special design or any deviation from the requirements mentioned.

(4) The rubrail shall be designed to resist the uniformly distributed load given in Table 11.8 when applied separately in the horizontal and vertical directions and shall be fabricated from austenitic stainless steel with a minimum 2.5% molybdenum composition, or non-ferrous material such as aluminium alloy. If non-ferrous components are used with steel fixings, insulation must be provided to prevent galvanic corrosion. Stainless steel, except those for fasteners, shall be Grade 1.4436 instead of Grade 1.4401 specified in the General Specifications for Civil Engineering Works.

(5) The traffic face of the rubrail shall present a smooth surface, free from sharp edges. Projections or depressions not exceeding 20 mm shall only be permitted at joints.

11.9 VEHICLE PARAPETS

(1) Vehicle parapets shall be designed to the containment standards given in Table 11.1 and to the strength requirements given in Table 11.6 in the case of reinforced concrete parapets. The limiting dimensions for metal and composite parapets are given in Table 11.7 and Figure 11.1 and Figure 11.2. The limiting dimensions for L4 reinforced concrete wall parapets are given in Figure 11.3.

(2) Combined metal and concrete parapets shall correspond in shape to the profiled concrete barrier as illustrated in Figure 11.2 with no kerb provided between the parapet and carriageway.

(3) The top of the parapets shall be shaped to prevent anybody from walking on the top of the high containment parapets located on general purpose roads. On limited access roads, different top shapes or metal top rails may be used.

(4) L3 and L4 parapets are generally strong for the containment of heavy vehicles but may be too stiff for light vehicles. If designs other than those incorporating a concrete profile as indicated in Figure 11.2 and Figure 11.3 are used, consideration should be given to include features in the parapet design to reduce the potential damages to light vehicles and injuries to passengers inside.

11.10 SIGHT DISTANCES

Sight distances are measured from a minimum driver's eye height of between 1.05 m and 2.0 m to an object height of between 0.26 m and 2.0 m, both above the road surface. A possibility therefore exists that the provision of shorter sight distances could sometimes be justified on the grounds that motorists can see through certain types of parapets. However, visibility through a parapet is liable to be obscured and distorted, and thus cannot be relied upon. A parapet of any kind shall accordingly always be treated as opaque for purposes of sight distance design.

11.11 RAILWAY OVERBRIDGE PARAPETS

11.11.1 High Containment Parapets

Railway vehicular overbridges shall be provided with L4 high containment parapets.

11.11.2 Overbridge Parapets

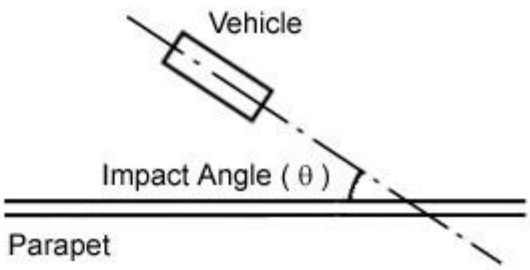
(1) Railway overbridge parapets shall have a minimum height of 1500 mm measured from the adjoining paved surface, and be of concrete construction with solid elevation and without applied finishes on the external faces of the parapets. New constructions of railway overbridge parapets shall have a higher minimum height of 1800 mm. They shall extend to a point not less than 8 m from the centre line of the nearest track, measured at right angles to the track.

(2) Notwithstanding the above minimum requirement, the height and extent of high containment parapets for railway overbridge shall be agreed among the railway authorities, project offices and the Chief Highway Engineer/Bridges and Structures prior to the design.

(3) Parapet copings shall be shaped so that persons cannot walk along them. Inner and outer parapet faces shall be smooth and free from projections or depressions that can be used as handholds or footholds.

Table 11.1 – Vehicle Parapet Groups

Level of Containment	Vehicle Impact Characteristics			
	Type of Vehicle	Vehicle Mass (tonne)	Impact Speed (km/hr)	Impact Angle (degree)
L1	Saloon Car	1.5	80	20
L2	Saloon Car	1.5	113	20
L3	Saloon Car	1.5	113	20
	Double-decked bus	22	50	20
L4	Saloon Car	1.5	113	20
	Rigid Heavy Goods Vehicle	30	65	20
	Articulated Heavy Goods Vehicle	38	65	20



Note 1 : The impact requirements of RHGV and AHGV should not be regarded as equivalent and no hierarchy is given between them. Both containments of RHGV and AHGV shall be demonstrated to ascertain that the minimum containment requirement of L4 is achieved.

Table 11.2 – Vehicle Characteristics

Vehicle Specification	Type of Vehicle			
	1.5 tonne saloon car	22 tonne double-decked bus	30 tonne Rigid Heavy Goods Vehicle	38 tonne Articulated Heavy Goods Vehicle
Mass (kg) Total vehicle mass Including dummy and Ballast	1,500 ± 75	22,000 ± 500	30,000 ± 900	38,000 ± 1,100
Dimensions (m)				
Wheel track (front to rear)	1.50	2.00	2.00	2.00
Wheel radius (unloaded)	N/A	0.55	0.55	0.55
Wheel base (between extreme axles)	N/A	6.80	6.70	11.25
Ground clearance of the front bumper measured at the corner	N/A	N/A	0.58	0.58
Number of axles (S = Steering Axle) (Note : Limit deviation ± 15 %)	1S +1	1S +2	2S +2	1S +4
Centre of gravity location (m)				
Longitudinal distance from front axle (CGX) ± 10 %	1.24	4.4	4.1	6.2
Lateral distance from vehicle centre line	± 0.08	± 0.10	± 0.10	± 0.10
Height above ground (CGZ)	0.53 ± 0.05	1.75 ± 0.10	1.90 ± 0.10 (for load only)	1.90 ± 0.10 (for load only)

Table 11.3 – Selection Guideline

Containment Level	Selection Guidelines
L1	For local access bridges on local distributors or rural feeder roads
L2	For bridges in general excluding those for which other containment levels are appropriate
L3	For bridges warranted by the scoring system in Table 11.4 or any special considerations deemed necessary by the designer and agreed by the Chief Highway Engineer/Bridges and Structures
L4	For railway overbridges, high risk locations or any special considerations deemed necessary by the designer and agreed by the Chief Highway Engineer/Bridges and Structures
Remark : Designers shall exercise judgment to consider the use of higher containment parapets where accidents risks are very high and the consequences of accidents are serious.	

Table 11.4 – Scoring System for Selection of L3 Containment Level Bridge Parapets

Road Characteristics	Criteria	Score
Speed limit	Speed limit $\geq$ 70 km/h	0.23
Height of road above ground or downhill slope	Height $\geq$ 20 m	0.19
Bus usage	Number of bus routes $\geq$ 10	0.19
Road geometry	Undesirable road geometry (See Note 3)	0.14
Traffic volume	Annual Average Daily Traffic (AADT) $\geq$ 30,000 (one-way)	0.07
Percentage of commercial vehicles	Percentage of commercial vehicles $\geq$ 20%	0.05
Features under road	Residents, schools, hospital or other similar occupants, or a water body, or expressways/ trunk roads exist in the vicinity.	0.08
Accident records	Frequent parapet impact accidents occurred (See Note 4)	0.05
	TOTAL	1

Note 1 : L3 containment level bridge parapets are warranted for a bridge section with a combined score of more than or equal to 0.70.

Note 2 : For individual assessment of score, a value of zero shall be adopted if the respective criterion is not satisfied.

Note 3 : Undesirable road geometry refers to a road section with radius less than 250 m for posted speed limit greater than or equal to 70 km/h, with radius less than 88 m for posted speed limit less than 70 km/h, with gradient greater than 8%, or at or within 20 m from junctions or interchanges.

Note 4 : Frequent parapet impact accidents refers to more than 10 accidents in 5 years. For new construction, accident records may be conservatively assumed to be frequent where the likelihood of such accident rate is high.

Table 11.5 – Parapet Heights

Parapet Type	Minimum Height (mm)
Railway foot-overbridge and underbridge walkway parapets	1500
Other pedestrian parapets	1100
Cycle-bridges including railway cycle-overbridge and underbridge bicycle parapets	1500
L1 vehicle parapets	1000
L2 vehicle parapets	1000
L3 vehicle parapets	1500
L4 vehicle parapets	1500
L4 vehicle parapets for new railway overbridges	1800

Table 11.6 – Strength of Reinforced Concrete Parapets

<table>
<tr><th rowspan="3">Item</th><th rowspan="3">Criterion</th><th colspan="4">Parapet Group</th></tr>
<tr><th rowspan="2">L1 & L2</th><th rowspan="2">L3</th><th colspan="2">L4</th></tr>
<tr><th>(1.5 m high)</th><th>(1.8 m high)</th></tr>
<tr><td>1</td><td>Minimum ultimate moment of resistance against vertical bending at base (reinforcement at traffic face).</td><td>25 kNm/m (intermediate section)

33 kNm/m (end section)</td><td>100 kNm/m (intermediate section)

133 kNm/m (end section)</td><td>165 kNm/m (intermediate section)

220 kNm/m (end section)</td><td>200 kNm/m (intermediate section)

265 kNm/m (end section)</td></tr>
<tr><td>2</td><td>Minimum ultimate moment of resistance against horizontal bending (reinforcement at outer face).</td><td>12.5 kNm/m</td><td>50 kNm/m</td><td>82.5 kNm/m</td><td>82.5 kNm/m</td></tr>
<tr><td>3</td><td>Minimum ultimate horizontal transverse shear resistance.</td><td>86 kN/m</td><td>220 kN/m</td><td>220 kN/m</td><td>220 kN/m</td></tr>
<tr><td>4</td><td>Minimum ultimate transverse shear load to be transferred at connected vertical joints between lengths of insitu parapet or precast panels.</td><td>66 kN</td><td>165 kN</td><td>165 kN</td><td>165 kN</td></tr>
<tr><td>5</td><td>Minimum ultimate moment or resistance of anchorage at base of precast panel.</td><td>37.5 kNm/m</td><td>--</td><td>--</td><td>--</td></tr>
<tr><td>6</td><td>Minimum thickness
- top
- bottom</td><td>
300 mm
300 mm</td><td>
300 mm
300 mm</td><td>
250 mm
675 mm</td><td>
250 mm
730 mm</td></tr>
<tr><td colspan="6">Notes : 1 For parapet with varying thickness or where the adjacent paved surfacing is not part of the structural element of the underlying structure, the base of the parapet shall be taken as any horizontal section not more than 300 mm above or below the adjoining paved surface.

2 The minimum ultimate moment of resistance against vertical bending shall reduce linearly from the base to zero at the top of the parapet.

3 The minimum strengths for L3 parapets are based on a parapet height of 1.5 m. Higher strengths shall be adopted for taller parapets.</td></tr>
</table>

Table 11.7 – Dimensions for Vehicle Parapets

Dimension	Description	Max. (mm)	Min. (mm)
a	Height of the top of the upper longitudinal member above the adjoining paved surface.		
	(i) L1 and L2 parapets	-	1000
	(ii) L3 and L4 parapets	-	1500
b	The height of the centre-line of the "Main" longitudinal member above the adjoining paved surface.	685	535
c	Clear distance between the longitudinal members, or between the top of the plinth and the longitudinal member above. The dimension is not necessarily constant within the parapet.		
	(i) between top 2 rails of L3 and L4 parapets	400	-
	(ii) between other rails	300	-
d	The overall depth of a longitudinal member.	-	50
e	Distance between the front face of a metal longitudinal member, or the top edge of a plinth, and the traffic face of the parapet.		
	(i) above the "Main" longitudinal member	± 25	-
	(ii) below the "Main" longitudinal member	- 25	0
	(Note : + ve towards traffic; - ve away from traffic)		
f	The distance between the traffic face of the parapet and the front face of the supporting post at its base, at whatever height the base may be.		
	(i) L2 metal parapets	-	150
	(ii) Other parapets	-	100
g	Width of the plinth.		
	(i) L2 metal parapets	-	500
	(ii) L1 metal parapets	-	350
h	Height of the plinth above the adjoining paved surface.	100	50
This table refers to Figure 11.1 and Figure 11.2.			

Table 11.8 – Minimum Design Loads for Pedestrian and Bicycle Parapets

Handrails and Rubrails	Other Rails	End and 90° Corner Posts		Other Posts		Infilling
		Parallel	Normal	Parallel	Normal	
0.7 kN/m	1.4 kN/m	1.4 kN	1.4 kN	1.4 kN	2.8 kN	1.0 kN

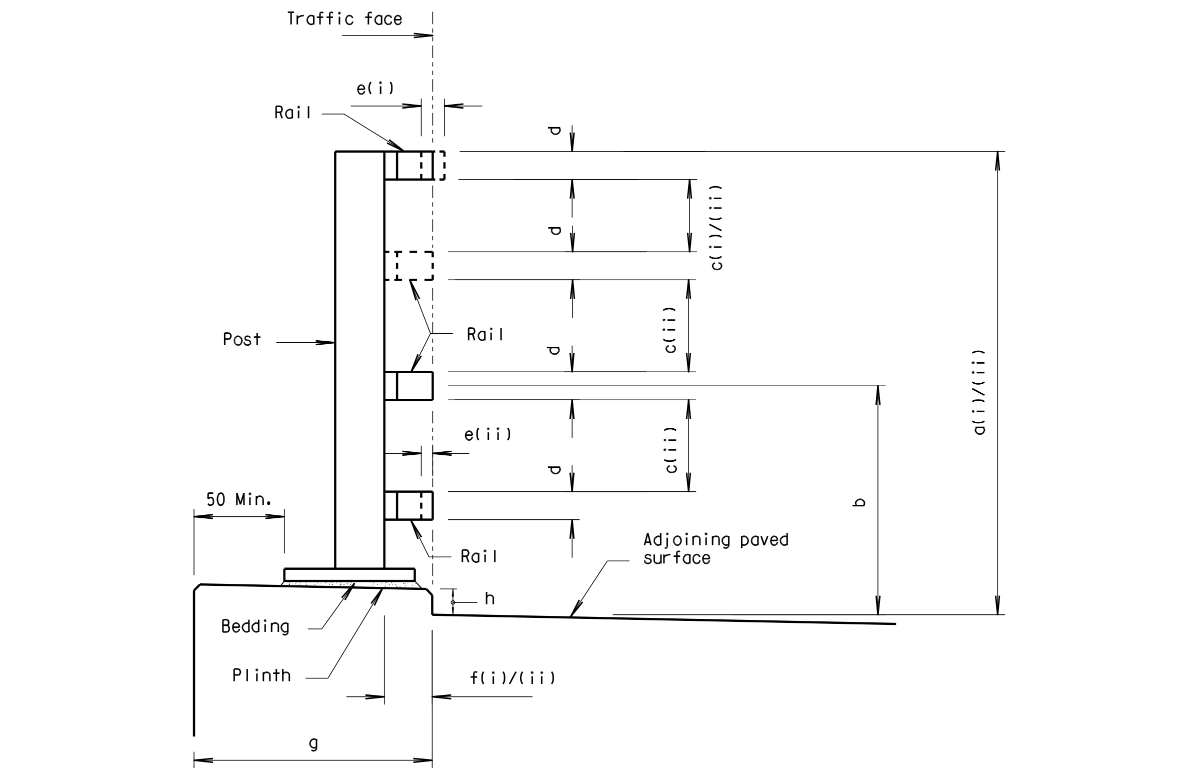
Table 11.9 – Dimensions for Pedestrian Parapets

Dimension	Description	Max. (mm)	Min. (mm)
a	Height of the top of the upper longitudinal member above the adjoining paved surface.	-	1100
b	The vertical distance between the top of the bottom longitudinal member and the bottom of the longitudinal member above.	-	800
c	Clear distance between the top of the plinth and the longitudinal member above.	100	-
d	Height of the plinth above the adjoining paved surface.	100	50
e	Horizontally measured gap between the infill members or between the infill members and the posts.	100	-
This table refers to Figure 11.4			

Table 11.10 – Dimensions for Bicycle Parapets

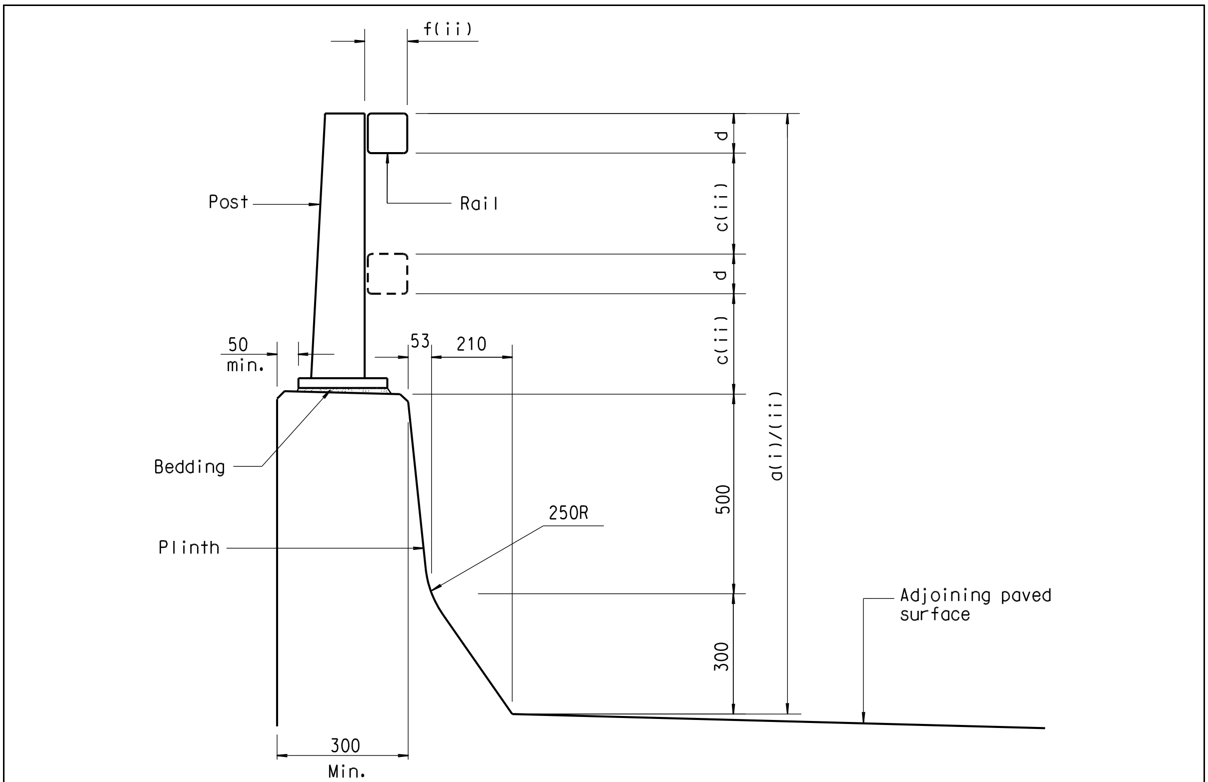
Dimension	Description	Max. (mm)	Min. (mm)
a	Height of the top of the upper longitudinal member above the adjoining paved surface.	-	1500
b	The vertical distance between the top of the bottom longitudinal member and the bottom of the longitudinal member above.	-	800
c	Clear distance between the top of the plinth and the longitudinal member above.	100	-
d	Height of the plinth above the adjoining paved surface.	100	50
e	The distance between the traffic face of the rubrail and the front face of the plinth.	100	75
f	Height of the top of the rubrail above the adjoining paved surface.	1100	1050
g	Horizontally measured gap between the infill members or between the infill members and the posts.	100	-
This table refers to Figure 11.5			

Figure 11.1 – Dimension of Vehicle Metal Parapets



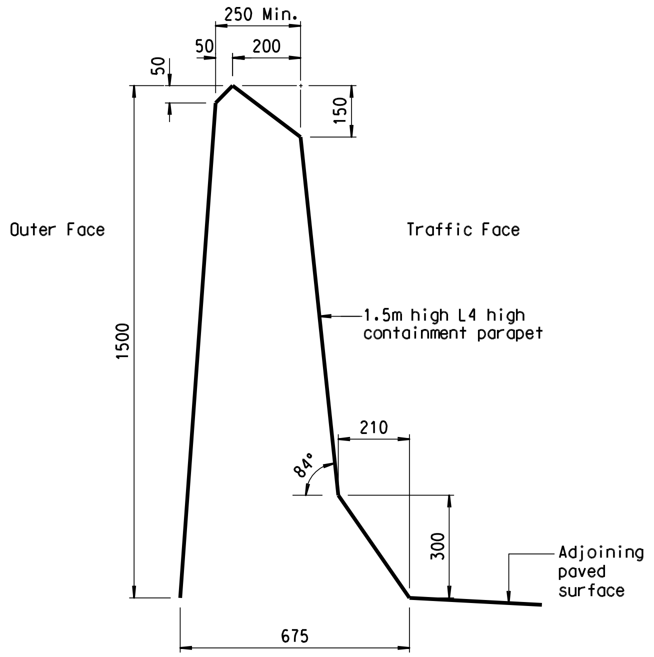
Note : See Table 11.7 for dimensions

Figure 11.2 – Dimension of Vehicle Combined Metal and Concrete Parapets

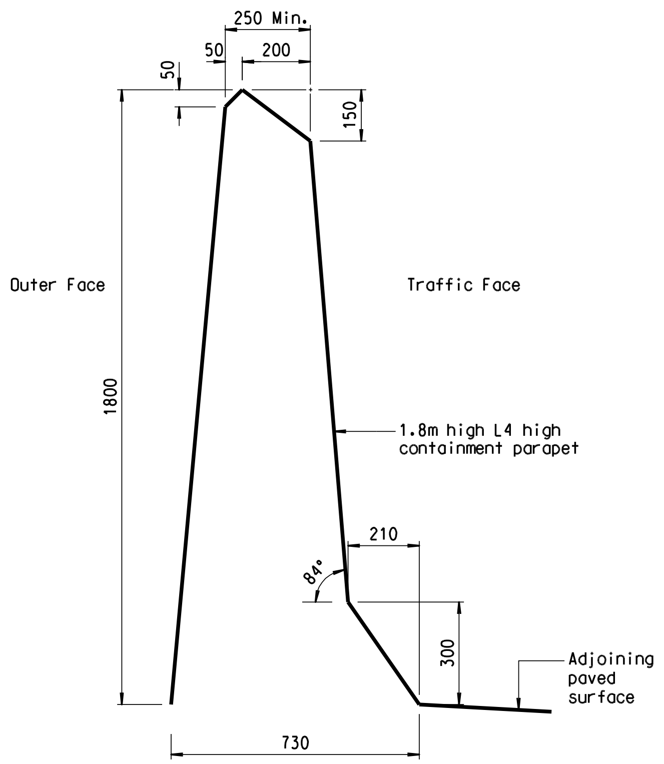


Note : See Table 11.7 for dimensions

Figure 11.3 – Dimension of L4 Reinforced Concrete High Containment Parapets

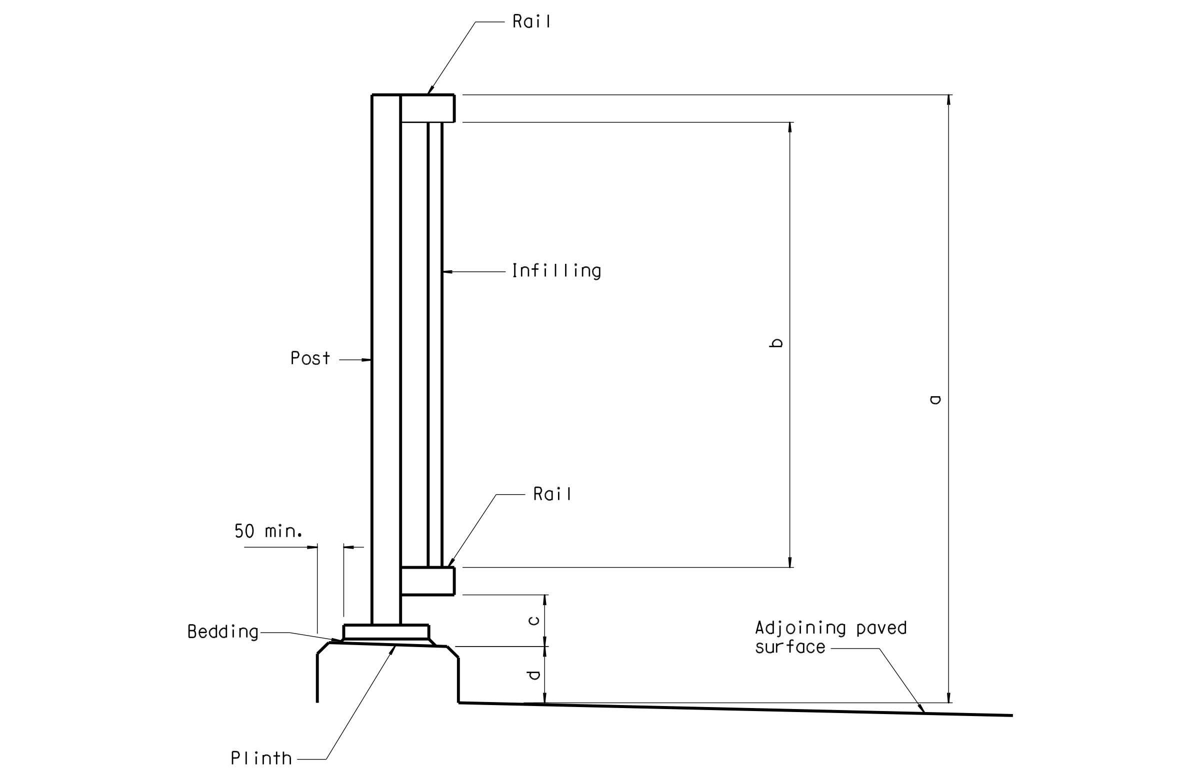


a) 1.5m High L4 Reinforced Concrete High Containment Parapet



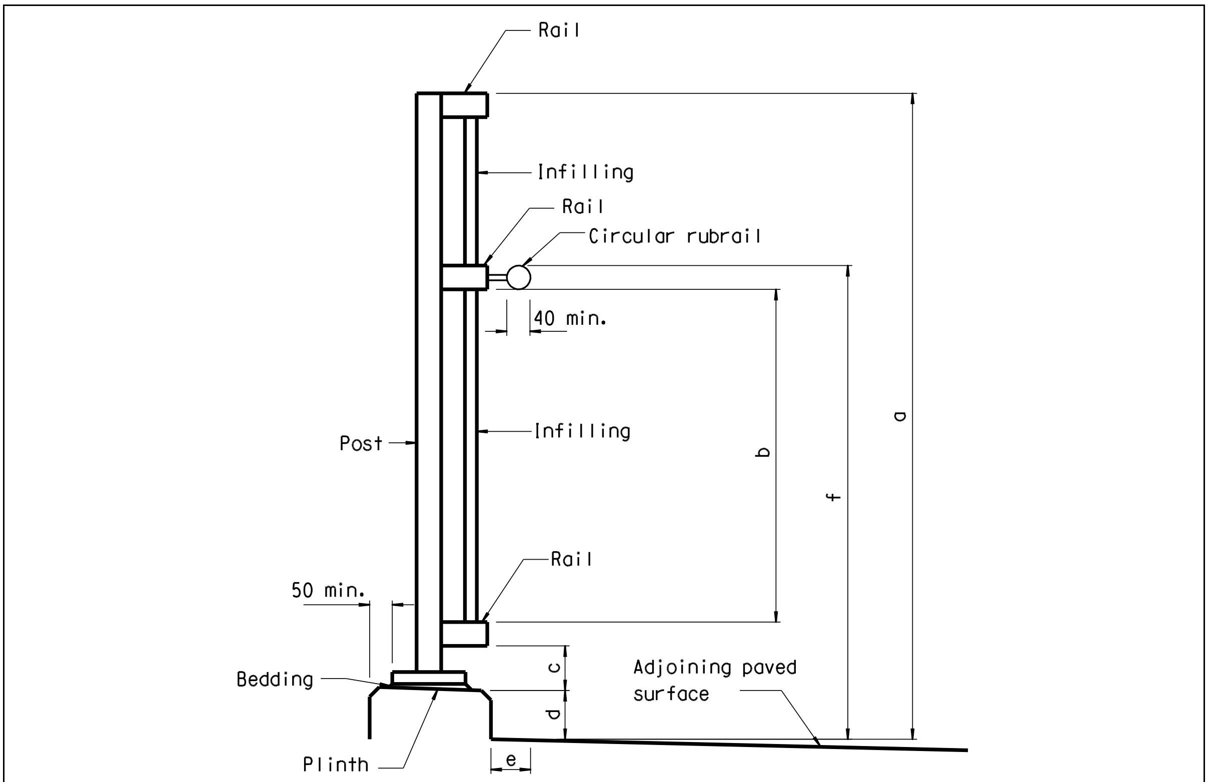
b) 1.8m High L4 Reinforced Concrete High Containment Parapet for Railway Overbridge

Figure 11.4 – Dimension of Pedestrian Metal Parapets



Note : See Table 11.9 for dimensions

Figure 11.5 – Dimension of Bicycle Metal Parapets



Note : See Table 11.10 for dimensions

CHAPTER 12 FOOTBRIDGES AND SUBWAYS

12.1 GENERAL

(1) A footbridge or subway scheme is not likely to be successful unless it meets the basic directional movements of the potential users and a study shall be made of existing and future movements of pedestrians before deciding on the best practical layout of the footbridge or subway. Access to footbridges and pedestrian subways shall be as short and direct as possible avoiding long detours when using ramps or stairs. Ancillary fittings such as pillar box for pump house, lighting and other electrical and mechanical installations shall be of minimum size, unobstructive and be incorporated into abutments or walls wherever possible.

(2) The provision of access for the disabled is a mandatory requirement. Apart from the requirements stated in this Manual, please also refer to the Volume 6 Chapter 8 of the Transport Planning and Design Manual (TPDM) published by the Transport Department. Access for the disabled shall therefore be included in highway crossing facilities where alternative access is unavailable. Ramps or lifts shall be provided in accordance with Transport Bureau Technical Circular No. 2/2000 issued by the then Transport Bureau. If in doubt, the advice of the Transport and Housing Bureau (THB) should be sought.

(3) The planning and layout design of pedestrian crossing facilities are normally carried out by the Transport Department. The designer shall make reference to the TPDM for the layout design. The layout design requirements contained in this Chapter are largely extracted from the TPDM for easy reference. Should there be any inconsistent requirements between this Chapter and the TPDM, the requirements stated in the TPDM shall prevail. Close liaison with the Transport Department at detailed design stage will ensure that consistent standards of provision are maintained.

12.2 COVERS

(1) All new permanent footbridges, elevated walkways, pedestrian subways and their associated ramps and stairways in the urban area shall be covered. In rural areas, the provision of covers depends on the circumstances of the particular location. Detailed guidelines for the provision of covers are given in the TPDM issued by the Transport Department. Applications for permission to omit covers where normally they would be provided should be made to the THB. Such applications shall contain appropriate justification, including background and reason for the request, and an account of the extent and result of any consultations with local interests, including the local District Officer. Also, a recommendation on provisions for future installation of covers should be made when submitting the application for omission of covers to the THB.

(2) When designing the roofs, consideration shall be given to avoid creating an uncomfortable environment during hot days. Measures shall be provided to prevent unauthorised access to the roof of footbridges for safety reasons. In a highly exposed and windy environment, due consideration shall also be given to minimising the effects of driving rains on pedestrians.

(3) Where there are security or privacy concerns, special measures, such as the use of screens, may be considered. The associated increase in wind loads shall be assessed and the affected neighbouring developments and maintenance authorities should also be consulted for consideration of such measures.

12.3 STAIRWAYS

(1) Stairs to permanent footbridges, elevated walkways and pedestrian subways shall have solid risers. Steps shall be levelled front and back.

(2) Risers should be not more than 150 mm high. This dimension may be increased to 165 mm high only in very exceptional circumstances where space is limited and where an alternative safe and convenient route for the disabled in the vicinity is available.

(3) Treads should be not less than 280 mm wide, but this dimension may be reduced to 250 mm if space is limited.

(4) Values for riser height "R" and tread width "T" shall satisfy the relationships :

$$580 \leq (2R + T) \leq 600$$

$$42000 \leq (T \times R) \leq 45000$$

The tread width "T" is the net tread width or "going". Riser faces may be inclined, but the extra tread width gained shall not be included in the value of "T".

(5) The number of risers in a flight shall not exceed 12, but may be increased to 16 if space is limited. The risers and treads of each step in a flight of stairs shall be uniform and risers shall not be variable in height over their width.

(6) The nosing of steps of staircases shall be in high colour contrast with adjacent surface to enable pedestrian including those with low vision to differentiate the steps. The flight of steps shall be kept free of long and straight horizontal pattern except for nosings.

12.4 RAMPS

(1) The gradient of pedestrian ramps should not be greater than 8.3% as such is the steepest gradient negotiable by a person in a wheelchair without assistance. Steeper gradient of up to 10% may be used where space is limited. Stepped ramps shall not be used even where space is limited without the approval of the Assistant Commissioner for Transport, Transport Department.

(2) The gradient of the centre line of circular ramps shall not exceed 10%. Ramps shall be provided with landings at vertical intervals of not more than 3500 mm wherever space and other considerations permit.

(3) The gradient of cycle ramps should not be steeper than 4%, but may be increased to 8% if space is limited.

(4) Experience indicates that the provision of traction strips is not preferred even on steep ramps, and the use of a non-slip surface will provide better skid resistance. Where traction strips have to be used, these shall be depressed. Embossed traction strips shall be avoided.

12.5 LANDINGS

(1) A user should be able to traverse a stairway landing in two comfortable strides. A stairway landing length of between 1500 - 1800 mm will enable most people to do this. Stairway landings may be reduced to a minimum of 1000 mm if space is limited.

(2) The length of ramp landings shall be not less than 2000 mm but may be reduced to 1500 mm where space is limited.

(3) The width of all landings shall be not less than that of the widest approach stair or ramp.

12.6 CHANGES IN DIRECTION

(1) The corner formed by a change in direction shall be splayed or curved to the largest radius that is practical. Pedestrian conflicts will thereby be reduced and feeling of insecurity eased. The latter is particularly important in subways, where pedestrians tend to feel vulnerable, and thus the corners should have a minimum radius of 4.6 m where possible in order that a minimum visibility distance of 4 m is achieved. This is also relevant if the subway is a combined pedestrian and cycle way and in these cases a greater visibility distance of 25m should be provided.

(2) The Commissioner of Police should be consulted on the need for addition convex mirrors in new subways for crime prevention.

12.7 DIMENSIONS

(1) Headroom shall be provided in accordance with the requirements of Chapter 13.

(2) The minimum clear width of walkway surface on footbridges, elevated walkways, and their associated ramps and stairways shall be 2000 mm, except on stairs to tram or similar platforms where a lesser width is necessary because of limited space. This width shall be not less than 3000 mm for subways and their associated ramps and stairways. Adequate allowance shall be made for railings/handrails, finishes, etc in calculating the clear width.

(3) To avoid impeding pedestrian movement, the clear distance between obstructions such as roof columns, parapets and handrails on footbridges, elevated walkways, subways, and their associated ramps and stairways shall not be less than 2000 mm.

12.8 PARAPETS AND HANDRAILS

(1) Parapets shall be designed in accordance with the requirement of Chapter 11.

(2) Solid parapets shall be used around subway ramps and stairways wherever the possibility exists of passing traffic splashing stormwater through parapet railings into the subway. However solid parapets should be used with discretion as they can produce an unpleasant sensation of confinement, particularly with narrow ramps and stairways.

(3) Where it is considered that there is a high risk of objects being dropped or thrown from the footbridge, consideration shall be given to full or partial enclosure of the crossing and its ramps or stairs. Normally, mesh infill will be suitable but if solid panels are specified, they should be translucent with provision made for cleaning. The design of the enclosure shall be such that unauthorized access to the sides or the roof covers is prevented. Similarly, consideration should be given to the provision of infill to parapets and step risers to protect the privacy of users and screening to protect the privacy of neighbouring dwellings.

(4) Parapets shall be at least 1100 mm high measured from the surface of the adjoining footway. In areas of high prevailing winds or where the headroom under a footbridge for pedestrian use only is greater than 10 m, the height of the parapet may be increased to 1.3 m.

(5) Handrails must be provided on both sides of all ramps and stairways, and consideration shall be given to the provision of central handrails on stairways 4000 mm wide or more. The handrails shall be fixed at a height of 850 mm above the nose of a step,as at greater height than this the elderly as well as the disabled have great difficulty in reaching the rail. At landings, handrails shall be set at a height between 850 mm to 950 mm above floor and extend horizontally not less than 300 mm beyond the first and last nosings of every flight of steps or beyond the ends of a ramp. In cross section, handrail shall provide a proper grip of 32 mm to 40 mm diameter.

(6) Handrails shall be fabricated from austenitic stainless steel or non-ferrous material such as aluminium alloy. If non-ferrous components are used with steel fixings, insulation must be provided to prevent galvanic corrosion. Stainless steel materials shall comply with Section 18 of the General Specification for Civil Engineering Works, except that Grade 1.4401 shall be replaced by Grade 1.4436 and stainless steel tube shall be Grade 1.4436.

12.9 DRAINAGE

(1) Subway floors shall be cambered to fall to each side at not less than 2.5% and shall be provided with longitudinal fall of not less than 0.67%. Gullies and slotted channels shall be provided at appropriate points to catch stormwater entering subways or underpasses, and convey it to the nearby stormwater drainage system, if necessary via a pump house provided for the purpose. To minimize the risk of blockage, an inspection pit with sump shall be provided at all changes in the direction of drain pipes in a subway/underpass. To minimize the risk of flooding due to trapped rubbish blockage, overflow weirs shall be provided at appropriate points in a subway/underpass. The

overflow weirs should be in a form of drainage inlet at the subway wall at a height slightly above the floor level.

(2) Footbridge roofs and decks shall be cambered to fall to each side and be provided with a longitudinal fall of not less than 0.67% unless otherwise approved by Chief Highway Engineer/Bridges and Structures.

(3) All pipe runs shall be capable of being rodded. An accessible rodding eye must be provided at each turning point in the drain pipe run. Rodding plug shall be fabricated of uPVC to facilitate maintenance. To facilitate rodding, stormwater drainage pipes for footbridges and subways shall not be smaller than 100 mm in diameter. Metal grating to surface channels, where provided, shall be of the easily replaceable hinged type and fabricated from cast iron or stainless steel.

(4) To avoid blockage of drain pipes, a corrosion resistant dome shaped grating or grille shall be provided at all inlets on roof covers. A recess or basin shall be provided around the grating for the water to flow in smoothly. The surface drainage system on covers adjacent to residential buildings shall be designed to minimize the chance of blockage by litter and be easily cleared.

(5) Structural steel and bare structural concrete components shall not be used as a drainage channel even for draining away occasionally seeping water. In hollow box beam decks, void drains shall be provided through the soffit at the lower end of the span. All parapet upstands shall be effectively sealed to prevent leakage. To reduce the chance of water staining the concrete surfaces, drip grooves shall be provided under the edges of the covers, decks, stairways, ramps and at transverse joints.

(6) During severe rainstorms, floodwater from adjacent catchments may enter the subways or underpasses via overland flow and accumulate there. In designing subways and underpasses including the capacities of associated water pumps, due consideration shall be given to minimize the risk of serious flooding due to overland flow from adjacent catchments. To reduce the risk of flooding due to malfunction of water pump(s), standby water pump(s) shall be installed as appropriate.

(7) The head of all stairways and ramps of subways shall be raised 150 mm above surrounding ground level to prevent the entry of stormwater. For stairways, an additional equal riser shall be incorporated. This shall contribute towards the total number of risers allowable in a flight. The additional step shall be sloped to the ground by means of a ramp with gradient complying with Clause 12.4. Ramps shall be extended to a level at least 150 mm above surrounding ground level and sloped down to the footpath at ground level. For low lying areas prone to serious flooding, consideration shall be given to raise the head of all stairways and ramps of subways even higher than the aforementioned as appropriate.

(8) The ground level lift entrance and the finished floor level of machine rooms shall be generally 150 mm above the adjacent ground level to prevent ingress of water and shall be ramped down to ground level. At locations prone to flooding or near the harbour side, the lift ground floor entrance and machine room finished floor shall be raised above the design flood level if practically feasible. The drainage details near the lift entrance at deck level shall also be designed to avoid ingress of water into the lift shaft.

(9) A proper drainage system, involving the use of pumps where necessary, shall be provided to prevent flooding of the lift pit and lift machine room from groundwater or rainwater. A water level sensing device shall be installed at the lift pit sump chamber. In the event of flooding being detected, the lift homing operation shall be activated and a fault signal shall be transmitted through the telemetry system to the remote monitoring centre.

(10) Slope within a pedestrian subway catchment area must be properly protected and drained so as to avoid the possibility of a washout of silt. Sand traps and grilles shall be provided wherever water is discharged into the surface channels of paved areas or into stormwater pipes to avoid flooding caused by blockage of the subway pumping system. Planting within a pedestrian subway catchment area shall be of the evergreen broadleaf type to reduce the amount of fallen leaves which can easily block drains and cause flooding. Catchpits shall have desilting sumps not less than 250 mm deep. They shall be covered and grilles shall also be provided to all pipe inlets to prevent large pieces of rubbish from entering and causing blockage of the subway drainage system.

(11) Requirements of flood warning system for subways or underpasses should refer to Clause 14.6.

12.10 LIGHTING

(1) Lighting schemes for pedestrian structures shall comply with the requirements of the Public Lighting Design Manual and shall be approved by the Lighting Division of Highways Department. The maintenance authority i.e. the Lighting Division and/or the Electrical and Mechanical Services Department shall be consulted in preliminary design stage.

(2) Light fittings shall be as inaccessible to pedestrian as far as possible and compatible with maintenance requirements. Lighting conduits and junction boxes shall not be surface-mounted except for steel structures.

(3) Where footbridges or subways are located in prestigious area, decorative lights may be considered in order to enhance the harmony of the environment. The Lighting Division of Highways Department shall be consulted at the earliest possible time.

12.11 ESCALATORS

12.11.1 Provision of Escalators

(1) The criteria for provision of escalators at footbridges and elevated walkways are detailed in Transport Bureau Technical Circular No. 2/2000. The agreement of the THB shall be obtained to the provision of escalators not complying with the foregoing criteria.

(2) Escalators should not normally be provided without an alternative means of ascent or descent, whatever the case may be, as during times of maintenance to the escalators the footbridge will be inoperable.

(3) Width of escalators can vary considerably, depending on the location, aesthetics, and other similar matters. However, escalators for footbridge should not generally have an effective width less than 1 m.

12.11.2 General Requirements

(1) As the Electrical and Mechanical Services Department is responsible for the maintenance of the electrical and mechanical parts of the escalators, the agreement of Director of Electrical and Mechanical Services shall be obtained at an early stage in the design of the escalators with respect to the details and requirements of the proposed escalators.

(2) Where escalators are provided, the level of the plinth at the foot of the escalator, protecting the mechanism, shall not be more than one step high, and shall be ramped down to ground level. This is because escalators are not particularly easy for the disabled to use, and this is made much more difficult when three or four steps have to be negotiated before reaching the escalator.

(3) Stainless steel materials shall comply with Section 18 of the General Specification for Civil Engineering Works, except that Grade 1.4401 shall be replaced by Grade 1.4436 and stainless steel tube shall be Grade 1.4436.

12.11.3 External Applications

(1) Escalators which are installed externally and are not fully sheltered by structures need more careful planning than escalators for internal use.

(2) Additional features required for external use include :

  (a) deeper anodising of aluminium components;

  (b) watertight floor pans and well drained external escalator pits;

  (c) air heaters on trusses;

  (d) cabinet for controller;

  (e) double varnish impregnation of electrical windings; and

  (f) neoprene covered multicore cables for electrical connections.

12.11.4 Inspection and Surveillance

Escalator installations need to be inspected frequently if lengthy interruptions of service are to be avoided. Rapid maintenance attention shall be available where escalators are provided at pedestrian grade separations, and the installation of special surveillance measures such as Closed Circuit Television (CCTV) or remote fault indicators should be given due consideration.

12.12 FINISHES

(1) Subway walls shall be tiled, to provide good light reflection and to discourage vandalism. Mosaic tiles shall not be used. Tiled surfaces shall stop at least 75 mm above floors and from external arrises, with the tile edges protected with a suitable hard material. Movement joints shall be provided for large tiles at 4.5 m intervals and at all construction joints in the structure.

(2) Ceilings and the pedestrian faces of parapet walls or beams shall be treated with washable and durable proprietary finishes. The pedestrian faces of parapet walls or beams may be tiled.

(3) Floor finishes shall be chosen to give adequate slip resistance. The slip resistance of surfacings shall be checked by the portable skid resistance pendulum tester developed by the Transport Research Laboratory U.K. (TRL). The surfacing, when new, shall have a slip resistant finish which has a skid resistance of not less than 65 TRL pendulum value under wet conditions.

(4) Stair treads shall be provided with non-slip nosings or special nosing tiles. Nosings shall be made conspicuous by the use of contrasting colours, or other means, so as to be clearly visible, particularly at night. Carborundum nosing strips are not recommended.

(5) Acrylic and polycarbonate sheetings shall be ultra-violet light resistant and have impact strength as Class A material complying with BS 6206.

(6) Components and fittings shall be fabricated from durable, corrosion-resistant material.

(7) If glass or fibre reinforced plastic units are used, the supply and installation of such shall be carried out by specialist contractors in the "Design, Manufacture and Installation of Glass (or Fibre) Reinforced Plastic Units" category of the List of Approved Suppliers of Materials and Specialist Contractors for Public Works.

(8) Joint gaps in walkway surfaces in excess of 12 mm in width shall have non-slip cover plates. All joints shall be sealed or caulked. Cover plates shall be set flush with the top of surfacing to prevent tripping.

12.13 WATERPROOFING

12.13.1 General Requirements

(1) Although good quality concrete is to all intents and purposes impervious, the construction joints and movement joints are always potential sources of leakage and thus shall be avoided as far as possible. Areas which are susceptible to water leakage shall always be waterproofed. Special attention shall be given to the design and detailing of the treatment at gussets, pipe flashings, upstands, movement joints and other awkward situations. The surfaces to be waterproofed shall be effectively drained to prevent ponding of surface run-off.

(2) Waterproofing shall be carried out by specialist contractors in the "Class II : Waterproofing of concrete surfaces" of the "Specialized Operations for Highway Structures" category of the List of Approved Suppliers of Materials and Specialist Contractors for Public Works.

(3) The main contractor engaged on projects involving the waterproofing of concrete surfaces shall be required to submit to the Engineer for approval full details of the waterproofing system he proposes to use. The details provided shall fully describe and illustrate installation of the proposed waterproofing system in the structure, and shall include details on substrate requirements, substrate ventilation if the waterproofing membrane is not vapour permeable, and treatment at gussets, pipe flashings, upstands, movement joints and other awkward situations.

12.13.2 Covers for Footbridges, Covered Walkways and Pedestrian Subways

Concrete covers shall be waterproofed. The waterproofing system shall have good adhesion to the substrate and be durable, colour fastness, UV resistance, easily repairable and capable of withstanding the impacts of regular maintenance and cleaning operations.

12.13.3 Pedestrian and Bicycle Subway Barrels

(1) Subway barrels, ramps and staircases below the ground surface shall be surrounded with an impervious membrane to ensure watertightness. Proprietary bituminous based membranes or similar shall be of heavy duty type and shall be installed in two layers to provide a total thickness of not less than 3.2 mm. A protective layer shall be provided to the waterproofing membrane to protect it from damage. Membranes on horizontal surfaces shall be protected with 50 mm of cement/sand mortar laid as soon as each section of membrane is installed. Membranes to vertical surfaces shall be protected with a 115 mm thick layer of brick or similar.

(2) Special attention shall be given to the design and detailing of the joint at the base of the wall as the reinforcement is normally congested at this location. The clear distance between inner layers of reinforcement shall not be less than 250 mm to facilitate concrete compaction. Reinforcement detailing shall be as simple as possible and due consideration shall be given to the positions of waterstops and construction joints.

(3) Continuous waterstops shall be provided at all construction and movement joints.

12.14 SPECIAL MATERIALS

(1) Acrylic and polycarbonate sheetings shall be ultra-violet light resistant and have impact strength as Class A complying with BS 6206.

(2) If glass (or fibre) reinforced plastic units are used, the supply and installation of such shall be carried out by specialist contractors in the "Design, Manufacture and Installation of Glass (or Fibre) Reinforced Plastic Units" category of the List of Approved Suppliers of Materials and Specialist Contractors for Public Works."

(3) Glaring of sunlight reflecting from roofing materials may sometimes attract complaint from the public. Designers should assess the effect of glaring of reflective materials on nearby residents and consider appropriate preventive measures where necessary during the design stage.

(4) Where glass is used, the risks and consequences of fracture and broken glass falling should be duly considered and appropriate mitigation measures should be taken. Amongst various measures, laminated tempered glass should be used as far as possible and should be heat soaked to BS EN 14179-1 with certification and adequate testing to reduce the risk of spontaneous fracture due to nickel sulphide inclusions. Fixings should be carefully detailed to avoid direct contact between the glass and other harder materials and undue thermal stresses.

(5) For overhead installations, the use of glass should be avoided as far as possible, particularly at blackspots and locations prone to fallen objects. Notwithstanding, the glass including the lamination if adopted should be subjected to enhanced inspections and maintenance, and other special requirements recommended by the suppliers upon opening of the concerned facilities to the public.

12.15 LIFTS

12.15.1 General Requirements

(1) As the Electrical and Mechanical Services Department is responsible for the maintenance of the electrical and mechanical parts of lifts, the agreement of Director of Electrical and Mechanical Services shall be obtained at an early stage with respect to the details and requirements of the proposed lifts. Apart from the requirements stated in this Manual, the designer shall make reference to the Design Manual – Barrier Free Access 2008 (BFA2008) published by the Buildings Department for the requirements of lifts. As BFA2008 is primarily used for design of new buildings or for alterations or additions to existing buildings, should there be any inconsistent requirements between this Manual and BFA2008, the requirements stated in this Manual shall prevail.

(2) To allow a wheelchair to turn inside a lift, the minimum internal lift car dimensions shall be 1500 mm by 1400 mm wide with a clear minimum door width of 850 mm. The clear height of the lift car shall be 2300 mm minimum.

(3) Floor finishes shall be non-slip complying with Clause 12.12. A tubular stainless steel handrail shall be provided on all sides (except sides with access door) of the lift car, extending to within 150 mm of all corners. The handrail shall be 950 mm above finished floor level.

(4) People waiting at lift entrance should not cause obstruction to the normal pedestrian flow. A lift waiting area with cover shall be provided at lift entrance.

(5) The horizontal clearance between a lift shaft enclosure and a road carriageway shall be in accordance with the TPDM. Barrier fences shall be provided and the lift structure shall be designed to resist collision loads.

(6) The lift shaft and lift car walls shall be glazed as appropriate to minimise visual impact and to facilitate security monitoring of the lift car interior. Glazing shall be clear toughened glass to provide resistance to damage by vandals and accidental breakage. Notwithstanding, lift glazing creates greenhouse effect and generates substantial energy demand for air conditioning and ventilation. To reduce the greenhouse effect, lift glazing shall be "low-emissivity" glass to lower the rate of solar heat transmission into the lift shaft and lift car. Due consideration should also be given to the following energy saving measures :

   (a) reduction of transparent areas subject to agreement by Highways Department, Transport Department, the police, etc but without jeopardizing the security monitoring of the lift car interior;

   (b) use of opaque walls on appropriate sides and orientating the remaining glazed sides to the south or east, where sunshine is less, if possible;

   (c) use of opaque roof top;

   (d) measures described in Clause 12.15.2;

   (e) other measures such as sun screen provision, as appropriate.

(7) The lower part of full-height glass panel in the lift car shall be translucent and provided with skirting.

(8) Access to clean interior surfaces of the glass panels of the lift shaft and lift car shall be provided.

(9) Provision shall be made for hoisting device including lifting beams or eyes within the lift shaft and the machine rooms. Lift machine rooms shall be above ground with good access for vehicles nearby and be provided with adequate artificial lighting and ventilation.

12.15.2 Ventilation

(1) At locations where the lift shaft is exposed to direct sunlight for long periods, mechanical ventilation of the lift shaft will be required and the provision of shading elements to reduce heat load shall be considered. Ventilation fans fitted with weatherproof louvres shall be installed at high level at the lift shaft wall to remove the heated air from the lift shaft enclosure. Ventilation fans shall not be installed at the roof top to avoid water inflow into the lift shaft. Stainless steel inlet louvres fitted with removable filters to trap dust particles shall be installed at low level in the lift shaft to permit replacement air to flow into the shaft.

(2) Ventilation fans shall be installed at the lift car ceiling. If air-conditioning is provided where situation warrants, a mechanical or an electrical drain system shall be installed to drain the condensation from the air-conditioning system.

12.15.3 Surveillance and Emergency Equipment

(1) A telemetry system shall be provided for transmitting lift fault signals automatically through the telephone network to a 24-hour manned remote monitoring centre, which shall contact the respective lift maintenance company to attend to the lift fault.

(2) An emergency alarm button, an intercom and a CCTV camera shall be provided in the lift car, which shall be connected to an intercom, a CCTV display monitor and an indication light placed at each of the lift entrances outside the lift. The emergency alarm button shall be connected to an alarm bell placed at the lift car top and an alarm bell placed at the ground floor lift entrance or lift pit.

(3) The intercom system shall comprise a 2-way speaker to allow people inside the lift car to communicate with people outside the lift at the landing call panel.

(4) A battery back-up system capable of maintaining a power supply to the emergency load for a minimum period of two hours shall be provided.

12.16 TACTILE WARNING STRIPS

Tactile warning strips for people with visual impairment shall be provided at the top, bottom and landings of ramps and staircases in accordance with TPDM Volume 6 Chapter 8 and also at lift entrances. They shall be made of durable and non-slippery materials and should contrast visually with the adjoining surfaces to provide clear indication of the routes to people with low vision.

CHAPTER 13 HEADROOM

13.1 GENERAL REQUIREMENTS

(1) Headroom shall be provided in accordance with Table 13.1.

Table 13.1 – Headroom

Type of Structure	Used By	New Construction (mm)		Maintained Headroom (mm)	
Overbridges Vehicle underpasses Footbridges	Vehicles	5100		5000	
Sign gantries	Vehicles	5500		5400	
Pedestrian subways Enclosed footbridges	Pedestrians	Length		Length	
		< 23 m	≥ 23 m	< 23 m	≥ 23 m
		2300	2600	2300	2500
Cycle subways	Cyclists	< 23 m 2500	≥ 23 m 2700	< 23 m 2500	≥ 23 m 2500
Tramway overbridges	Trams	5600		5600	
Light Rail Transit System overbridge	Light rail vehicles	6200		5800	
Railway overbridge	Train	7000		7000	

(2) The headroom to be provided is the effective headroom after compensation for vertical curvature and deflection as described in Clauses 13.3 and 13.4.

(3) The headroom specified for new construction includes an allowance of 100 mm for subsequent resurfacing. Maintained headroom in Table 13.1 includes no such allowance, and must be preserved at all times thereafter.

(4) The specified headroom shall be provided over the widths prescribed for horizontal clearance in Table 13.2, as well as over carriageways and hard strips or shoulders.

Table 13.2 – Horizontal Clearance

Design Speed (km/h)	Height of Obstruction (mm)	Minimum where crossfall slopes towards obstruction at: ≯ 2.5% (or away) (mm)	≯ 4% (mm)	> 4% (mm)
50 or less	< 3000	500	600	600
	≥ 3000	500	600	800
		Minimum (mm)	**Desirable (mm)**	
Above 50 and less than 80	< 3000	600	1250	
	≥ 3000	1000	1250	
80 and above	Any	1000	1500	

(5) During construction of overhead structures across carriageways, maintained headroom clearance may be reduced to 4700 mm with adequate signing.

13.2 MEASUREMENT OF HEADROOM

(1) Headroom shall in general be measured vertically. In cases where the combined crossfall and longitudinal gradient of the road passing under a structure exceeds 4%, headroom shall be measured at right angle to the road surface at the point of minimum clearance.

(2) Headroom shall be measured from the lowest point of the overhead structure. The lowest point shall be taken as the lowest lighting fixture, sign, signal or similar protrusion rather than the lowest part of the overhead structure itself.

13.3 COMPENSATION FOR VERTICAL CURVATURE

Where the vertical alignment of a road passing under a structure results in a sag curve beneath the structure, the headroom shall be increased in accordance with Table 13.3. The sag radius is the mean radius of vertical curvature over a 25 m chord length measured along the carriageway.

Table 13.3 – Compensation for Vertical Curvature

Sag Radius (m)	Additional Headroom (mm)
< 1000	100
1000	80
1200	70
1500	55
2000	45
3000	25
6000	15
> 6000	nil

13.4 COMPENSATION FOR DEFLECTION OF STRUCTURE

Allowance shall be made for the effect of dead load deflection on headroom design. In addition, the headroom provided shall be increased to allow for the maximum deflection that can occur at mid-span due to live load.

13.5 COMPENSATION FOR SIGNAL AND OTHER INSTALLATIONS

At locations where signals, lighting and other equipment related to operational service are to be installed, suitable allowance shall be made in the headroom design. Such installations shall never be permitted to intrude into the headroom provided.

13.6 TRAMWAY AND LIGHT RAIL TRANSIT OVERBRIDGES

Although Table 13.1 gives a value of 5600 mm for the headroom to be provided over tram tracks, this figure, which refers to the Hong Kong Tramways Company's double-deck trams, is given for general guidance only. A figure of 6200 mm has been quoted as suitable headroom for Light Rail Transit (LRT) tracks, but this again shall only be used for general guidance and it may be possible to achieve a lower headroom. The appropriate authority shall always be consulted as to the precise headroom to be provided under a highway structure crossing a tramway or a LRT track.

13.7 RAILWAY OVERBRIDGES

Wherever a highway structure crosses a railway track, the appropriate railway authority shall be consulted as to the precise horizontal and vertical clearances to be provided. For general guidance, a minimum of 7 m clearance above highest rail level so as to avoid any interference with the overhead energy supply equipment shall be provided.

13.8 HIGHWAY STRUCTURES ACROSS DRAINAGE CHANNELS

For any highway structures or utilities hung on bridges over existing or planned drainage channels, consideration shall be given to ensure that the soffit level is above the designed water level with adequate freeboard. In addition, consideration shall be given to the effect of the backwater tidal flow on the final designed level. The agreement of headroom should be obtained from Drainage Services Department at the early stage of the design.

CHAPTER 14 STORMWATER DRAINAGE

14.1 GENERAL

(1) Drainage design shall be in accordance with the Guidance Notes No. RD/GN/035 “Guidance Notes on Road Pavement Drainage Design” published by the Highways Department and the Stormwater Drainage Manual published by the Drainage Services Department. Requirements of drainage design for footbridges, subways and underpasses are detailed in Clause 12.9.

(2) Stormwater drainage installations have often caused problems. The main reasons are :

(a) small diameter pipes, which have become blocked;

(b) changes of direction in pipe runs, which have become blocked;

(c) poor detailing of intakes and outlets resulting in frequent blockages;

(d) inadequate provision for clearing blockages; and

(e) thin-walled pipes broken by pressure of wet concrete, or incompletely sealed pipe joints through which cement grout has entered during construction, so that pipes have been blocked permanently before they have ever functioned.

14.2 PIPES AND PIPE LAYOUT

14.2.1 Minimum Diameter

Stormwater drainage pipes for vehicle and pedestrian/bicycle highway structures shall not be smaller than 150 mm and 100 mm in diameter respectively. Longitudinal carrier drains shall be provided with rodding eyes at interval not more than 20 m.

14.2.2 Material

(1) Drain pipes shall be of uPVC, unless some good reasons make the use of an alternative material desirable. uPVC drain pipes shall comply with BS 4660, JIS K-6741, or a comparable acceptable national standard, and have dimensions similar to those given in Table 14.1.

Table 14.1 – uPVC Drain Pipes

Standard	Nominal Size (mm)	Approximate Inside Diameter (mm)
BS 4660	110	103
	160	152
JIS K-6741	100	107
	150	154

(2) Joints in uPVC drain pipes may take the form of either :

  (a) elastomeric sealing ring joint assemblies, or

  (b) solvent welded joint assemblies.

(3) The holding down and fixing arrangements of the clamps and hangers for the drain pipes onto the structure shall be fabricated from austenitic stainless steel. Stainless steel materials shall comply with Section 18 of the General Specification for Civil Engineering Works, except that Grade 1.4401 shall be replaced by Grade 1.4436 and stainless steel tube shall be Grade 1.4436.

(4) Galvanized mild steel fixing arrangements may only be used for internal fixtures.

14.2.3 Layout

(1) Drainage layouts shall be considered at the preliminary design stage so that a suitable scheme for an inconspicuous pipe layout can be worked out and incorporated in the design from the start. All general arrangement drawings shall include drainage proposals.

(2) Drain pipes shall be installed with as few changes of direction as possible. Where changes of direction are unavoidable, an inspection chamber, inspection cover, rodding eye or other means of access shall be provided so that any blockage can be cleared easily. Rodding plug shall normally be of uPVC, unless some good reasons make the use of an alternative material desirable.

(3) Because uPVC pipework is light, internal pipework can become displaced or damaged during concreting, particular care shall be taken to ensure that uPVC pipework to be cast into concrete is firmly fixed within the formwork before concreting.

(4) Structural steel and bare structural concrete components shall not be used as a drainage channel even for draining away occasionally seeping water. In hollow box beam decks, void drains shall be provided through the soffit at the lower end of the span. All parapet upstands shall be effectively sealed to prevent leakage. To reduce the chance of water staining the concrete surfaces, drip grooves shall be provided under the edges of the structure and at transverse joints.

14.3 MOVEMENT JOINTS

(1) As a general principle, pipework shall be designed to accommodate the relative movement of the structures at both sides of the movement joint.

(2) Vertical or near-vertical pipework crossing from a deck to a column shall be split, with the upper pipe discharging into a hopper head at the top of the column downpipe large enough to accommodate both the anticipated discharge and movements. Flexible connections are unlikely to last as long as the rest of the structure and shall be avoided as far as possible, or be accessible and replaceable.

14.4 INTAKES

(1) All stormwater intakes on highway structures shall be provided with sumps, which can be cleaned quickly and effectively by mechanical means.

(2) The sumps shall be at least 250 mm deep. This depth of the sump may be greater than the depth of the deck required from purely structural considerations, and may necessitate the provision of a fascia or similar to screen or disguise the drainage installations.

(3) With short structures, stormwater intakes can usually be located in the approaches, thus avoiding the need to provide very large construction depth for sumps.

(4) Stormwater intakes shall have a grating in the plane of the carriageway and a side inlet overflow weir behind it, so that if the horizontal grating is blocked by rubbish, the vertical inlet can still operate.

(5) Sags on viaducts are prone to flooding when intakes are blocked by rubbish carried to them by stormwater. Emergency outlets shall accordingly be provided at all sags to discharge floodwater reliably under such conditions.

14.5 OUTLETS

Stormwater drainage system on highway structure shall discharge directly into a manhole and then be connected to the nearby at grade stormwater drainage system.

14.6 FLOOD WARNING SYSTEM

Flooding of subways or underpasses could cause serious threat to public safety and there is a need to incorporate a flood warning system in subways or underpasses that could give maintenance authority a flood warning in any flooding incident. The flood warning system comprises an “on-site” Monitoring Unit. The Monitoring Unit shall be able to detect flooding incident and send a telephone message to the message receiver of the maintenance authority through mobile networks. For detailed requirement of flood warning system, reference shall be made to the Practice Note No. BSTR/PN/002 issued by the Highways Department. Flood warning system shall be provided for all new subways or underpasses vulnerable to flooding. For cases where the chance of flooding is remote and there is no need for such warning system, agreement from the maintenance authority shall be sought in advance.

CHAPTER 15 AESTHETICS

15.1 GENERAL

(1) Aesthetics of the built environment is of increasing importance as we are in one way or another all influenced by visual qualities of our surroundings. Bridges and other highway structures are usually in large scale, highly functional artefacts with long service lives, located where they are frequently seen and experienced by the public. A good appearance is therefore one of the functional requirements for such structures.

(2) Good appearance however is not simply a matter of elegance in the structure itself since it invariably forms part of a broader scene. A structure only “looks” well if it is appropriate to its setting. Consequently, good design aims to achieve total harmony of the structure with its local environment and setting. Structure shall be designed with due consideration on the built environment of today, and with a view towards the future environment during the lifespan of the structure. Where appropriate, consideration shall also be given to design a local theme on Hong Kong’s traditional, historical or cultural background.

(3) Aesthetic appreciation of structures tends to be made from two distinct perspectives: the dynamic viewpoint or the viewpoint of motorists/users using the structures directly; and the static/dynamic viewpoint of observers contemplating such structures while not actually utilizing them. This differing perspective exists to some extent for any/all sculptural artefacts, but is particularly relevant to structures due to their prominence within the local landscape. The utility of this aesthetic fundamental is that both points of view need particular attention during the design process.

(4) The introduction of structures invariably brings about changes in the local environment. Structures shall always be designed with an aim to provide for pleasant aesthetics whilst maintaining rational engineering principles (such as structural simplicity and effective form) and efficient function, taking operation and maintenance requirements into due consideration.

15.2 PRINCIPLES AND APPROACH TO AESTHETIC DESIGN

(1) Aesthetics development process shall allow for freedom of design. This Chapter is therefore advisory in nature. It will guide the designer through the methodology and intuitive process to arrive at a pleasing design.

(2) Aesthetic design involves the fundamental consideration of a number of key elements which are listed below. The inter-relationship between the principal elements is illustrated in Figure 15.1.

(a) Global Measures
- Functional consideration
- Structural consideration
- Design theme
- Context and environmental aspects

- Long-term appearance
- Maintenance and operational requirements

(b) Aesthetic Elements
- Transparency and slenderness
- Form
- Proportion
- Scale
- Expression of function
- Unity and harmony
- Visual stability and balance
- Rhythm and rhyme
- Illusion

(c) Detailed Effects
- Light and shade
- Texture
- Colour and chromatic design
- Lighting highlight

(d) External or Ancillary Features:
- Landscaping
- Ornamental features
- Drainage
- Existing structures
- Noise barriers and enclosures
- Lighting and signage

These key elements are discussed in the following paragraphs with particular reference to highways structures. Similar discussion will apply to railway structures.

15.3 GLOBAL MEASURES

15.3.1 Functional Consideration

(1) The purpose of a bridge or subway is to overcome obstacles as experienced by vehicular traffic, pedestrians (including the disabled), cyclists or any other users. A retaining wall will withstand forces exerted by the retained ground which has been re-shaped to give room for the road carriageway. Similarly, a noise barrier shall perform its function of reducing noise at the identified sensitive receivers. These basic functional requirements shall never be compromised by the aesthetic design.

(2) In addition to being visible to the public, structures are physically experienced when used. For instance, drivers and passengers in vehicles ‘experience’ the quality of carriageway surfacing and deck configuration, the safety of the parapets, the flickering effect of nearby objects such as noise barriers and tunnel walls, and the distant views. Pedestrians ‘experience’ the quality of materials, finishes, proportions, heights, movements, sounds and colours. These experiences and how people respond to them are called ‘human factors’. Design including aesthetic consideration of the general

arrangement should aim to achieve a positive response from intended users by taking full account of the human factors.

15.3.2 Structural Consideration

(1) Apart from fulfilling its functional purpose, an aesthetically pleasing structure shall clearly express its structural behaviour. The main structural components shall form the primary lines and dominate the appearance so as to properly communicate the structural behaviour to the viewer and user.

(2) Giving a structure an attractive appearance can always be achieved through the expression of characteristics by the structure itself.

15.3.3 Design Theme

Where there is a series of highway structures proposed along a route alignment, or a number of highway structures contained in close proximity within the same visual envelope, or there exists a number of ground features, a design theme shall be developed to guide the design of the structures. Elements created shall have common theme, but they can vary somewhat in response to different aesthetic opportunities of the particular structure. This will allow the development of strong visual ties between the highway structures along the route and contribute to enhancing the surrounding visual environment. An approach for the design theme is to study on the local traditional, historical and cultural aspects so as to identify the desirable common features for application at the structures.

15.3.4 Context and Environmental Aspects

(1) Highway structures may be designed as strongly dominating or as hardly noticeable, depending on the intention of the design and to suit the site condition. The design of the context requires careful consideration and should be appropriate for its specific location or the structure will look odd and out of place. The structural system shall integrate into the site environment. In this respect, the character of the surrounding environment shall be identified and categorised. Typically, the location and setting of highway structures are broadly classified into the following categories:

- Urban or rural;
- Commercial or industrial, high or low density residential;
- Old or modern; and
- Hilly terrain, open or built up area.

(2) The system, form and scale of the highway structures, and the materials selected shall complement to the surrounding environment in order to visually and functionally form one unit of a character which will reflect the local landscape and setting.

(3) Character gives a bridge an identity that has a deliberate effect on people. The character shall match the surroundings and even alert people about where they are. If the bridge is big, e.g. a cable stayed or suspension bridge, it will become a landmark with an

imposing character of its own. But, if it is a small bridge in a built up area, its prime value may be to blend in with its local environment. However, the size of a bridge is not the only determining factor. In some cases, a footbridge can be given a character to be a sculptural statement in its environment and act like a local landmark.

15.3.5 Long-Term Appearance

(1) Effort shall be made to ensure that attractive structures remain so for their anticipated life. This can be achieved by using durable materials that are resistant to weathering and prolonged usage, and do not significantly deteriorate in time. Less durable materials, if used at all, shall be adopted in components which can readily be maintained or replaced.

(2) Sensible detailing to reduce the chance of subsequent spoiling of surfaces by natural stain and mould growth, accidental damage or deliberate vandalism is essential to maintain long-term appearance. The initial and long-term appearance of structures will be greatly enhanced by, and is largely dependent upon, carefully supervised construction practice to achieve a sound workmanship, as well as a systematic program of regular maintenance thereafter. Figure 15.2 to Figure 15.7 show some examples of good detailing.

15.3.6 Maintenance and Operational Requirements

(1) Highway structures are designed to have long service lives. It is therefore prudent that the aesthetic design will not unnecessarily increase the cost of long term maintenance. A balance on the aesthetic needs, functional requirements and constructability shall be optimised with the future maintenance.

(2) Aesthetic features shall not add significant difficulties to the maintenance requirements of the structure, but shall rather be compromised with the functional requirements of other components. The choice of materials, positioning, designed form and access arrangement shall be carefully considered to ensure that the aesthetic features and other parts of the structure can be safely inspected and maintained.

(3) Design detailing will also affect maintenance as illustrated below :

(a) near-horizontal surface may be changed to an inclined surface to minimise the dampening of the horizontal surface and its adjacent vertical faces, to deter the growth of fungi, or to direct wash-off away from vulnerable places;

(b) joints shall be suitably designed, sealed, positioned or covered to prevent leakage which will cause nuisance; and

(c) drip grooves with suitable sizes and numbers, may be positioned to cut off water flow so as to reduce the possibility of staining and leakage.

(4) If non-traditional or unusual feature, such as coloured concrete is used, the design shall provide for regular maintenance of the quality appearance.

(5) Where green planting is incorporated, consideration shall be given to the use of automatic irrigation and gravity drainage system. For planting proposed on roof structure or other similar locations, adequate access and other safety provisions shall be provided to enable subsequent maintenance works to be carried out. The extent of irrigation and access provisions shall be agreed with the maintenance authority. Any irrigation pipe works along the structure shall be properly covered up, and due consideration shall be given for water leakage from the pipe.

15.4 AESTHETIC ELEMENTS

15.4.1 Transparency and Slenderness

(1) Transparency gives a feeling of lightness and weakens the visual barrier effect of a bridge as shown in Figure 15.8. Transparency is influenced by the bridge substructure, piers, columns, arches, cables etc, and for low bridges, by the depth of cross sections and girders.

(2) For tall bridges, transparency is best obtained by using discrete columns. Round columns or multi-faceted columns will create a more slender appearance. The alternative use of narrow rectangular columns may give a stable look while maintaining an open view from the transverse direction. Provision of an opening in a column will also achieve transparency and in the extreme case, twin columns will have very good effect in giving a slender appearance.

(3) For low and narrow bridges, transparency is best obtained by using single columns. However, for low and wide bridges, the preferred solution, both aesthetically and structurally sound, is often the use of two slender circular columns or streamlined portals at each support. For low bridges, the use of circular columns is considered suitable since the small relative height of the bridge structure would reduce the visual punching effect which may be apparent for tall bridges.

(4) The slenderness of a bridge is normally represented by the ratio of the span length to the cross section depth, or alternatively to the cross section width. This ratio can vary greatly with the viewing angle. To look slender, a bridge should have a cross section that is not too deep when viewed from, say, a 30º angle. The visual slenderness of the cross section of a bridge can be increased by use of a horizontal overhang, chamfer or fillet for the cross section. The visual slenderness is also very much affected by the bridge parapet. Open type parapet and solid wall parapet would have different effect on it.

15.4.2 Form

(1) All forms are perceived as volumes dressed in light and shadows, or as profiles and silhouettes. They are perceived as single volumes at distances, and as compositions of sub-volumes at closer range. Each sub-form communicates with neighbouring forms so that they together make up a composition of space. This means that a bridge is visually linked with its surroundings, such as nature, roads, other structures and buildings, and

that when passing under or over a bridge the experience is characterized by the play of these forms and their composition into a whole.

(2) Traditional bridge forms include horizontal and vertical lines and planes, arches and cables as principal structural modes. Some of these are illustrated in Figure 15.9 to Figure 15.11. There are many variations in these basic forms, such as girders, trusses, cantilevers, towers and so on, embracing a whole range of shapes. For long structures, horizontal and vertical curvatures add further dimensions to the total impression of form.

(3) The global shape of highway structures tends to be linear, with the individual lines and planes acting like boundaries for the spaces defined by their volumes. The lines can be straight or curved, and the planes similarly flat or curved. However, too many lines will result in disorder and confusion for the observer. The design objective is often to create lines and planes, to create integrated and unbroken continuities.

(4) The choice of form is an essential prelude to any design, and shall be seen to be appropriate to the function and situation of the structure. The form chosen will depend on whether the structure crosses a waterway, a road or a valley, and where its supports can be economically founded, among other factors.

15.4.3 Proportion

(1) Proportion may be defined as the scheme of dimensional ratios that will produce a desirable form or assembly of associated forms.

(2) The proportion of a structure strongly influences its character. For example, exaggerated height, as in church architecture, can induce an air of reverence or awe. There are however, limits to the range of acceptable distortions, and certain dimensional ratios are widely accepted as being more pleasing than others.

(3) The celebrated "golden ratio" A:B = B:(A+B) (that is 1:1.618) is a rectangular ratio known for its pleasing effect. It has been widely used to define the proportions of anything from windows to entire buildings. Unfortunately, such simple ratios have little relevance to more complex arrangements of shapes, and good design can never result solely from the application of mathematical formulae without the influence of a creative imagination and a sensitive feeling for what is good.

(4) The principal proportions of a bridge are governed by the ratio of pier height to span, width to span, and superstructure depth to span using the dimensions shown in Figure 15.12. In the final analysis, the designer should acquaint himself with the effect on proportion of varying these ratios by personal observation. Sensible and precise observation of things that look right or wrong in the everyday scene is the real key to good judgement in such matters.

(5) Where there is an "assembly of associated forms", proportion is as much concerned with appropriate relationships between them as with their individual proportions. A poorly proportioned structure may have components which appear too light or too heavy for their apparent role, leading to suggestions of structural deficiency, imbalance

or lack of stability. Such errors often result from reliance upon two-dimensional drawings only. For example, a section through a typical column and deck will give a completely different impression of proportion to the real thing, as may be seen in Figure 15.13, where the great area of the underside of the bridge is more apparent as perspectives come into play.

(6) The total configuration of a three-dimensional object, such as a bridge, is difficult to appreciate from two-dimensional drawings and the use of scale models, computer generated or hand-sketched 3D renderings to verify initial concepts is a virtual necessity for most but the simplest of designs.

15.4.4 Scale

(1) Scale is concerned with size relationships, but in terms of visual effect, it also has much to do with relative extravagance or exaggeration in the choice of dimensional detail. The quality of scale has been described as one of the most potent tools in the art of juxtaposition of scenic elements. Figure 15.14 shows how the sizes of the texture patterns will affect the appearance of a retaining wall.

(2) Taking a bridge as an example, considerable variations in apparent scale can be achieved by choosing solid parapets instead of open ones, and by selecting multiple slender columns rather than single massive supports. The structural elements can also be subdivided in a manner similar to what was frequently done in classical architecture. By subdividing into odd numbers, they often tend to appear more interesting.

(3) Where a large structure can be viewed as a whole, its successful integration will depend very much upon its relationship with other scenic elements of similar scale, such as any adjacent visual connections, major topographical features or the road itself.

(4) Where a large structure is likely to be viewed at close quarters, the scale and texture of its components become more important, and their relationship with correspondingly smaller local features will require greater attention.

(5) By virtue of its size, a bridge will invariably be a significant element within the visual envelope and more often than not, it will need to be "scaled down" if it is not to dominate its setting. For this reason, it is frequently necessary to design urban bridges with as slender a profile as can be reasonably achieved or with other techniques to hide the massive appearance.

15.4.5 Expression of Function

(1) The whole purpose of a highway bridge is to conduct traffic over an obstacle, and this function can best be expressed by a smooth, flowing appearance. Aspects of this function are illustrated in Figure 15.15. Highway curvature is indeed one of the bridge designer's greatest allies in the achievement of appropriate visual form, when given sympathetic structural treatment. The unattractive appearance produced by constructing a sharply curved bridge with a series of straight beams as an example of the opposite effect is illustrated in Figure 15.16.

(2) In general, function shall be clearly expressed using minimum means, as shown in Figure 15.17 for example. The selected structural element for expression of function will attract much attention and shall be aesthetically pleasing itself. Bearings for space frame structure are quite often highlighted to spell out the support function but bearings for medium to short span bridges are normally concealed. It is difficult to make a movement joint aesthetically attractive and usually they are concealed.

(3) In principle, there shall be no contradiction between external form and internal function. Each part of a structure shall be seen to be clearly capable of fulfilling its apparent role even if its form is modified by other considerations. For example, a column which derives its stability entirely by fixity at the base, shall not be designed with a narrow base and excessive flare towards the top, as this would apparently contradict its function. However, provided the column base is given reasonably robust dimensions to express sufficient fixity at that level, there would be no objection to an increase in width towards the top, particularly where this modification is utilised for the better positioning of bearings. Figure 15.18 illustrates these points.

15.4.6 Unity and Harmony

(1) Unity of form and harmony are important and are largely related to the simplicity and refinement of design. Something extra to interest the eyes of the observers is often needed. An illustration is given in Figure 15.19.

(2) A harmonious relationship exists between a number of things when they complement each other so that their combined effect is more pleasing than their separate contributions.

(3) The achievement of harmony in adding a structure to a landscape or townscape will at first sight seem to be complicated because of the interplay of diverse shapes and colours in the surroundings, many of which are beyond the control of the designer. The problem is simplified, however, if only the more significant scenic elements are considered and if the number of novel features is kept to a manageable minimum by repeating selected shapes, colours or textures already present in the scene.

(4) A structure should present a stable, simple and elegant appearance, in harmony with the surrounding landscape or townscape. This means that there should be no discordant features and some of the structure’s attributes, such as form, rhythm and colour, shall blend in a positive way with correspondingly important characteristics in the surroundings.

15.4.7 Visual Stability and Balance

(1) For visual stability, particularly when viewed from a passing vehicle, a bridge requires a sufficient measure of verticality. The apparent inclination of sloping supports may change from different angles of view, giving the impression that decks are slipping off, or piers falling over, as the observer travels by. Other inclined members can intensify the effect. Even when viewed from a static viewpoint, trapezoidal supports used on a long curving bridge can give the impression of varying in shape or tilting at different

angles. This effect is illustrated in Figure 15.20. Inclined members are not inherently visually unstable, but designers must be aware of the visually unpleasant possibilities attendant on their use.

(2) An even number of superstructure modules is often held to give rise to what is known as “unresolved duality”, which is akin to visual instability in that such arrangements tend to lack composure and unity, as illustrated in Figure 15.21. In a twin-arched bridge, for instance, the eye will wobble from one centreline to the other, producing a feeling of restlessness.

(3) Another unpleasant effect will be produced when a central pier coincides with the highest point of a bridge superstructure, so that the deck seems to droop away from that point, as illustrated in Figure 15.22. The effect is not only a loss of unity but is also similar to placing a column under the midpoint of an arch, which is self supporting, and therefore associated with contradiction of function. There is an evident need for a structure to look not only stable and balanced, but also reasonably logical if it is to please the eye.

15.4.8 Rhythm and Rhyme

(1) Rhythm is concerned with the organisation of repetitive features, which shall as far as possible be both uniform and simple. Repetitive features shall also relate to other rhythmic details in the vicinity. Thus, the spans of a viaduct or multi-span bridge shall be equal, or follow a constant rhythmic pattern such as a constant span-to-height ratio. If, as often happens, a road viaduct follows a similar route to a railway viaduct, the appearance of both will be enhanced if the latter follows the span rhythm of the former.

(2) A common arrangement, occurring locally at the old Ap Lei Chau bridge, is for a number of short approach spans to lead up to a single longer navigation span, usually with shorter anchor spans on each side, as shown in Figure 15.23. Such an arrangement rarely presents a satisfactory appearance because the navigation span is long enough to break the structure’s rhythm without being long enough to dominate the whole concept.

(3) Figure 15.24 represents a 19th century viaduct which presented a very satisfying rhythmic appearance. Good rhythm has also been achieved at the footbridge spanning over the Shing Mun River in Sha Tin, where the use of a constant rhythmic pattern results in a satisfying appearance as shown in Figure 15.23.

(4) Rhyme requires closely related repetitive forms be compatible. In an idealised Roman aqueduct shown in Figure 15.24, the arches of the higher tiers sub-divide regularly the arch spans of the lower tiers, thus visually ‘rhyming’ with them. A common shortcoming with inexperienced designers is to overlook the need for rhyming so that, say, the columns of a footbridge roof do not ‘rhyme’ (that is, follow a span multiple) with the columns of the main structure, or the posts of the parapet railings do not ‘rhyme’ with the roof columns.

(5) Varying ground levels along the length of a bridge or vertical alignment of the structure can cause drastic linear variation of ground to soffit dimensions. To create pleasing rhythm and rhyme for multi-span bridges where the voids underneath each span can be

seen at distance, designers shall try to maintain similar proportions for the dimensions at each space as bordered by the columns, ground and soffit.

(6) Repetition of regular structural features or finish treatments, similar to the use of rhythm, can also be used to create visual stimulation. This applies to both major structural elements such as bridge piers, or minor features such as bridge parapets, lighting poles or finish treatment for a stretch of roadside retaining walls. For highway structures, this concept of repetition is often used to develop various design themes of a particular portion of the highway as well as to evoke a "sense of place" for the visually sensitive receivers.

(7) Neglect of proper rhythm and rhyme makes an irregular, confused impression on the observer who feels instinctively that anything so ill-organized as the structure he is observing cannot perform effectively. This is particularly applicable to the design of noise barriers and enclosures, where the designs are derived from parameters and conditions that are often completely separate and different from those of the adjacent highway structure. The orderly appearance of a rhythmically designed structure, whose components rhyme in disciplined fashion, conveys a comforting impression of strength and efficiency.

15.4.9 Illusion

(1) Illusion can interfere with visual perception and, if the designer is to avoid unexpected distortions in the appearance of his structures, he must study the effects with the often use of scale or computer models, and plan to overcome them.

(2) The solution often lies in what may be termed 'counter illusion', which is the deliberate distortion of form to oppose anticipated adverse effects caused by the primary illusion. For example, a well known illusion is that long horizontal spans appear to sag, and a deliberate upward camber will create the necessary counter-illusion. Figure 15.22 shows another illusion of apparent sag. Vertical walls often appear to lean outwards at the top, and they can be given a slight batter to offset this illusion. Similarly, the entasis or swelling on classical columns to counter the illusion of mid-height narrowing under certain lighting conditions, are legitimate architectural devices. Such measures enhance the apparent stability of a structure.

(3) As another example, a subway set at a gradient and emerging through an angled headwall will appear to have a distorted profile and the solution is to level off before the exit or use a strictly perpendicular headwall.

(4) The designer shall be aware of illusion, either to exploit or counteract its effects in the interest of good design.

15.5 DETAILED EFFECTS

15.5.1 Light and Shade

(1) The proportions of edge beams, cantilevers and parapets shall be chosen so that the shadows thrown by them onto the structural elements below emphasize the form of the structure, and do not cause either the structural elements or the shadows themselves to appear disjointed or mis-shaped.

(2) The valuable contrasting effect of light and shade tends to be less pronounced in Hong Kong than in higher latitudes where shadows are longer and there is a lower relative intensity of reflected light. Nevertheless, by use of, for example, long deck overhangs, the dominating effect of a substantial deck height can be visually reduced in the shadow as illustrated in Figure 15.25. The shadow cast onto the fascia girder by the deck overhang diminishes the prominence of the fascia girder by visually concealing it. When the visually diminished fascia girder is contrasted with the highlighted surfaces of the parapet and deck fascia, these latter elements stand out by comparison. This effectively increases the apparent slenderness by focusing the visual attention on the relatively slender elements, the parapet and deck fascia.

(3) Other possible ways to manipulate light and shadow include :

(a) change the inclination of the fascia girder such that it receives less light;

(b) use white cement or different texture to give equivalent emphasis; and

(c) add horizontal bevels, grooves along the parapet to highlight the appearance and thus draw attention away from the fascia girder.

15.5.2 Texture

(1) Surface texture can have a significant effect on appearances and shall always be carefully selected. Different textures may be used in combination on the same structure in order to modify apparent proportions, to provide contrast and interest, or to emphasize the different roles of structural components such as abutments and superstructure. Surface textures are often formed in the process of manufacturing, processing, cutting, shaping and/or positioning of materials. By skilful application of textures, additional interest can be added to structural forms as shown in Figure 15.26 to Figure 15.27.

(2) Texture can be used to create localised shadows of a darker colour. Coarse texture will create shadowing effects that can be seen from distance.

(3) Textures shall be used where they are effectively appreciated. Little will be gained by using textures on short piers facing traffic on an expressway, since all traffic pass by at high speed. Tall piers are however, more noticeable and their long faces can often be broken up by textures. Similarly, alternating smooth surfaces with textured ones can break up long wall effect.

(4) Large areas of smooth, fair-faced concrete shall be avoided since such areas are not only difficult to form without blemishes, but also tend to emphasize rather than conceal any minor defects. Furthermore, they weather badly. Such surfaces could instead be made less insipid by treating them with grooving (strategically planned to coincide with construction joints, if present), ribbing or other texturing.

(5) Small smooth surfaces can also have textural value. Concreting shall be carefully controlled to avoid forming marks. Special forms can also be used to create both linear and pattern type textures.

(6) Rough surfaces may look stronger and are most suitable for elements that carry heavy loads, for example piers and abutments, while smooth surfaces are more suitable for other elements, for example exposed beams, girders and slender columns.

(7) Broken-ribbing has the advantage of making graffiti-writing and bill-posting difficult; simple off-the-form ribbing is cheap and relatively effective in many situations; bush-hammering is suitable for relatively narrow surfaces, such as parapets, but is expensive and sensitive to concrete defects; exposed aggregate textures, whether by wash-and-brush or sand-blasting techniques require careful control of concrete quality and aggregate content for uniformity, and are best if used on precast cladding panels rather than on large cast-in-situ areas. Special mould linings in rubber or other materials can give interesting results. The weathering of textured surfaces and the action of rain washing dust over surfaces, or of fungi growing on damp areas, will have a great influence on long term appearance and should be carefully considered in relation to each unique situation.

(8) Concrete textures can rarely be forecasted accurately on the drawing board, and the designer shall always consider the need for mock-up panels on site (not merely fixed samples for approval), so that he can achieve the most suitable results by trial and error, paying particular attention to techniques at joints and corners.

15.5.3 Colour and Chromatic Design

(1) Colours, whether they are applied or natural, play a major role in aesthetics. Colours can be applied to structure surfaces by painting or staining, or through the selection of appropriate constituents such as various aggregates, cements and admixtures as integrated into concrete. The greyish colour of concrete is by itself a relatively unobtrusive tone. Since being unobtrusive helps to make a bridge more visually acceptable, structural concrete shall not be coloured under normal circumstances. This has the added benefit of avoiding unnecessary maintenance commitment since painting on concrete requires regular maintenance.

(2) Particular attention shall be paid on choice of colour. Earth tones are known to perform better, while striking strong colours shall be avoided since they are known to be more easily broken down by sunlight and environment. Colours can be used to strengthen or lessen the visual effects of individual members. Warm colours (red, yellow, brown, etc) will emphasize the size of forms while cool colours (blue, green, purple, etc) can reduce the visual importance of elements. Bright colours make bold startling aggressive statements and shall be used with caution, while soft colours will easily blend into the

surrounding environment. Maintenance issue shall be addressed for accidental wearing, e.g. vehicle impacts on barriers, etc.

(3) Colour can either be used to blend a structure with its surroundings or to create a contrast between man-made objects and nature. While a bridge of natural concrete colour will have slight deviations in the shade of grey concrete for various components (due to different concrete mixes , etc), it tends to be the human perception that a bridge with applied colour will have a better look.

(4) Nevertheless, the application of colours in a structural design composition is considered to be an important tool in achieving both functional and environmental enhancement. It is particularly useful and often essential where associated human factors need to be incorporated. For instance, interior surfaces shall be ‘inviting’, ‘comfortable’ and ‘safe’ instead of ‘forbidding’, ‘bleak’ and ‘vulnerable’. Surfaces shall also be ‘neat’, ‘warm’ and ‘inspiring’ instead of ‘messy’, ‘cold’ and ‘depressing’. If these factors fail to be properly addressed, structures are likely to detract from, rather than to contribute to, the quality of an environment into which they are introduced. Use of architectural cladding can be considered for applications at certain prestigious locations, with due consideration to longevity and maintenance issues.

(5) The most difficult part of developing a functional as well as pleasing chromatic design is the management and control of subjectivity or personal likes and dislikes in the selection of colours and finishing materials. It is therefore essential to the success of a scheme to adopt an objective approach in the lead-up to its design.

(6) To this end, three critical aspects of a project are identified which call for action in step with normal design development stages, as follows :

(a) *The Chromatic ‘Mood’ of a District and/or Local Environment.* This ‘mood’ is catalogued as part of the design investigation stage by recording the chromatic composition of the environment, bringing together all visible colours exactly as they are perceived by observation. From this record, the designer determines whether there is something missing from the chromatic make-up of the area that shall perhaps be introduced to stimulate interest or build up local character.

(b) *Primary and Secondary Functions of a Structure.* Primary functions include the purposes for building it whereas secondary functions include any perceived purposes arising from its location on a site and in a district such as pinpointing the ‘genus loci’, traffic route and direction of travel. The detailed analysis of all functions determines the range of human factors to be considered, which will be much wider for structures designed for pedestrian use than for vehicular movement.

(c) *The Design of the Structure Itself.* The basic structural form represents the solution to a number of identified engineering problems within a framework of known site constraints. It may not in itself fulfil all the requirements imposed by its functions. Certain elements may need to be highlighted or obscured to either reinforce or suppress (soften) basic design features. A thorough understanding of the structure and the role played by each of its component parts, including their effect and relation to the chromatic design, provides a ready framework for adjusting visual quality to serve attendant functions using the chromatic design tool.

(7) Only after taking adequate stock of these elements is a designer equipped to develop a practical design theme and consider a palette of colours from the natural colour spectrum which would best serve to achieve his design objectives. Figure 15.28 shows an example.

(8) For chromatic design, one of the most comprehensive codes available today is the Swedish Standard No. SS 019102, which adopts a Natural Colour System (N.C.S.) for identification, selection and specification of the complete range of visible colours. A comparable colour identification system in common use is the Pantone Colour System. However, no system will adequately address in every detail finishing aspects such as matt, high gloss, metallic or textured, etc, making it necessary at times to refer to manufacturers' publications such as charts, product specifications and samples in order to complete a scheme design.

(9) Long term colours cannot be forecasted precisely on the drawing board. Appearance will vary with a number of factors including material characteristics (absorption, roughness, metallic or non-metallic, etc). The designer shall always consider the need for controlled pre-construction experiments on site in the actual lighting conditions (not merely fixed samples for approval) so that trial panels can be viewed in order to select the most suitable material.

(10) In the final analysis, the success of a design will be contingent upon the appropriateness of its theme, its interpretation in the overall layout and the degree to which objectives have been met in the detailed treatment of individual elements.

15.5.4 Lighting Highlight

(1) The appearance of a bridge shall be considered for the daytime as well as the night time hours, with respect to pedestrian and vehicular traffic, and based on a pleasant appearance at a distance. It shall be noted that maintenance is an issue if additional lighting beyond what is required for trafficking is proposed, but for structures that are focal points and will become local or regional landmarks, it can be a desired option.

(2) A well thought out lighting plan is important. The design will dictate the outlook of the highway structure, especially during night time. Interior and exterior lighting require a coordinated plan where surrounding structures contribute to the overall night time streetscape rather than compete individually. Lighting shall first achieve the functional tasks and not be obtrusive to users and the general public. During day time, the lighting fixtures shall appear visually congruous with the surroundings.

(3) The following aspects shall be considered functionally: adequate lighting for the safety of traffic, and sometimes for navigation guidance of air traffic near tall bridges, and pleasant ornamental lighting especially with respect to slow moving traffic such as pedestrians and bicycles. Lighting can also be provided to cast certain elements of a bridge into shapes and textures such as barriers, piers and columns, abutments and

towers of larger bridges. Figure 15.29 illustrates how lighting add aesthetic value to a highway structure.

(4) For subway entrance, footbridge and walkway, efforts shall be made to provide a transparent or translucent roof cover. It will allow penetration of natural light, evoke a sense of openness and reduce lighting costs.

(5) With respect to the luminance level of functional lighting for highway structures, reference shall be made to the guidelines provided under HyD's *Public Lighting Design Manual*. During retrofitting, upgrading or major maintenance works to highway structures, due consideration shall be given to improving the standard of luminance levels in line with the recommendations under *Public Lighting Design Manual*, where appropriate, to facilitate the disabled and improve security to users.

15.6 EXTERNAL OR ANCILLARY FEATURES

15.6.1 Landscaping

(1) Landscape element is an integral part of the highway structure that needs to be considered in the initial stage of design process. A landscape section forming part of a design memorandum shall be developed in the project initiation stage, with specific landscape goals, assessment criteria to measure its effectiveness, and guidelines for landscape and surface treatments.

(2) Integration of the highway structure into its surrounding landscape is one of the most important consideration in aesthetic design. The creation of wastelands under overhead structures due to a lack of light, access preventative treatments or other inability to utilise these spaces has an adverse effect on the aesthetic quality of the structure and shall be minimised.

(3) Provision of soft landscape in form of "greening" is of increasing demand from the general public. Designers shall explore the opportunity for incorporating soft landscape and planting facilities onto structures and in its vicinity to enhance the visual and living quality of the whole environment. Regarding the provision of permanent planters and irrigation systems on future footbridges and flyovers, reference shall be made to Environment, Transport and Works Bureau Technical Circular (Works) No. 10/2005.

(4) Soft landscaping, in particular tree and shrub planting, have the benefits to :

(a) anchor the structure on the ground plane;

(b) soften the scale and extent of hard surfaces;

(c) screen parts of structure;

(d) add amenity value to the local area;

(e) provide landscape focus to features;

(f) add visual interest to the landscape; and

(g) stabilise earth slopes/surface.

(5) Soft landscape plays an important role, both to mitigate the visual impacts of the highway structure and to establish a distinctive character for the visual envelope. In general, the plant species selected shall possess a particular form, colour, and texture. Large mature trees shall be planted where practicable, to provide a dramatic vertical expression to set a backdrop for the visual environment and contrast the horizontal lines that typically dominate a structure. Planting shall be placed in a gradually layered manner to open up the roadway space. To create a more natural environment, repetition of planting monotony shall be minimised. The resulting profile shall undulate and vary somewhat in elevation along the highway structure.

(6) Creeper/vertical planting, shrubs and ground cover shall be adopted to reinforce the basic landscape theme, and to add colour and interest. On extended routes, combinations of shrubs may be used to develop identities along different sections of the route to assist recognition by the motorists and develop a sense of arrival to areas of significant prominence.

(7) An alternative way of increasing the level of perceived soft landscape is “borrowed landscape” which refers to the use of existing planting outside, but adjacent to the highway structure. This will suit situations where the actual available area for planting at the highway structure is severely limited or considered impractical for reasons of irrigation and maintenance access. An example of this would be to utilise existing tree planting behind a transparent noise barrier to soften the visual impact of the motorists.

(8) An integrated landscape design enhances the visual appearance of structures considerably and shields up the less attractive parts of the structure. In the aesthetic design of structures, the following aspects need to be addressed :

(a) provision and choice of soft and hard landscape on and around structures;

(b) maintenance requirement;

(c) engineering consideration;

(d) visual continuity to local streetscape/townscape; and

(e) future development and flexibility for change in landscape design.

(9) Hard and soft landscaping including planting, decorative lighting, furniture, and architectural features at structures provide a more enjoyable experience for pedestrians and road users. In addition, soft landscaping provides an orientation that is frequently needed to visually navigate complex urban environments by enhancing roadway delineation, screening undesirable elements and separating incompatible land uses. They also aid in improving environmental conditions through buffering of dust, noise and reducing glare.

(10) Plant species with low demand of water supplement and reasonable resistance to pests and disease infestation shall be selected. Plant groups with similar levels of maintenance requirements shall be placed together. Requirements of irrigation systems and maintenance access shall be agreed with the maintenance authority.

(11) Placement of soft and hard landscape elements shall not obstruct the motorists' sight lines and visibility splays as recommended in the Transport Planning and Design Manual (TPDM) Volume 3, Section 3. The requirements under the TPDM Volume 6, Chapter 8 shall also be referred to with respect to the provision of suitable physical constraints, such as planters, to areas underneath highway structures where headroom is less than 2 m.

(12) Special consideration shall be given to landscape treatment under elevated roadways, to integrate these areas with adjacent pedestrian and/or planted areas and not to leave them as sterile and unpleasant environments. Landscape design shall integrate all the hard and soft landscapes into a perfect matrix that visual obstruction to other fixtures such as fire hydrants and traffic signage will not be caused. In particular, 1.5 m all-round clearance should be maintained at the fire hydrants and the surrounding ground shall be formed lower than the hydrant pit cover to ensure that any emergency operation and the like will not be hindered.

(13) A highway structure shall integrate harmoniously with its surrounding environment, townscape or streetscape to achieve a visual continuity. This is particularly important for grade separated pedestrian facilities within urban environments, since the size and scale of the structure will often allow the structure to fit wholly within the visual envelope and form a major visual marker within the local townscape or streetscape. Reference shall also be made to the Streetscape Master Plan of the respective District, if any, as well as checking for any streetscape enhancement works being planned in the vicinity of the highway structure, and which style/type of enhanced paving and street furniture are to be adopted in such works. A co-ordinated landscape design to tie in with the pre-set theme under the Streetscape Master Plan shall be developed in such a way that harmony within the entire visual envelop is not disrupted.

(14) Generally, the costs associated with hard and soft landscape works are usually a small percentage of the overall capital costs. Given the value of environmental contribution and visual effectiveness of these measures, landscaping is a cost effective way of improving the appearance of highway structures as well as providing a more pleasant environment for the public users.

15.6.2 Ornamental Features

(1) Typically, an aesthetically pleasing design is more often than not a simple design expressing the function of the highway structure by clear presentation of its singular and collective components through interaction of form, scale and proportion. Simple forms and uninterrupted lines will create attractive bridges. Visual features shall derive from sensitive detailing of structurally required elements and the extensive use of artificial ornamental elements shall be avoided or generally be limited to where genuine need for these elements is identified.

(2) There are many opportunities by which the sensitive detailing of a highway structure can create visually attractive ornamental features. Typically, this pertains to the choice of form and treatment of various structural components. Some examples include the followings :

(a) use of a single shape family for all elements of the piers and vary its proportions for multi span bridges of varying heights;

(b) use of bevelling or tapering of the surfaces of the pier cap end;

(c) use of pilasters to form vertical lines to juxtapose the horizontal flow of the bridge deck;

(d) battering the front face of the abutment wall to create a dynamic visual perspective to motorists passing below the bridge deck; and

(e) horizontal bevels on the vehicular parapets to accentuate the horizontal lines of the bridge and reduce apparent vertical height of the parapet.

(3) Where artificial, non-structural ornamental features are adopted, the designs shall be simple so as not to draw the focus of the user from the overall appearance of the highway structure. The material selected shall be durable and possess a long service life. The detailing of the ornamental feature shall facilitate future replacement necessitated by either general maintenance or change of presentation theme of the highway structure. However, they shall not be easily detachable by any one passing by.

15.6.3 Drainage

Drainage is an integral part of the highway design. For ease of maintenance, unsightly pipework is often attached on the external faces of structures. This practice is not desirable. Pipe layouts shall instead be inconspicuous. The need to avoid (or clear) potential blockages and maintenance requirements may make exposed pipelines necessary in most situation, and consideration should be given to conceal or mitigate the intrusiveness of the pipes. Drainage details shall be integrated with the aesthetic design of the highways structures. Special drainage pipe sections or materials, or shield cladding may be incorporated to give a more attractive appearance. Suitably detailed drainage elements can also help to provide a sense of scale and rhythm to the highway structure. Other aspects for consideration when carrying out the drainage design are detailed in Chapter 14.

15.6.4 Existing Structures

(1) Special aesthetic considerations apply when retrofitting elements to existing structures (e.g. adding lifts to footbridges) or when upgrading or undertaking major maintenance of existing structures. Existing pleasing aesthetics shall be maintained and preferably enhanced. However, aesthetics cannot simply be ‘added’ onto an existing structure, but should be considered in the context of the whole environment.

(2) This also applies to the situation when a new structure is to be located next to an existing one. For example, a new bridge is to be built next to an attractive historical bridge. The best design is perhaps not an exact copy of the old bridge. Cost, design standard, new construction methods available and the weathered condition of the old structure would tend to rule that out. A solution may be a copy of the basic form of the old bridge, but using new and different construction methods and materials. The intention might be to tie the two structures together visually so that the old bridge still looks elegant, as does the new one because of its refined simplicity. Both structures shall look as if they belong to the site.

15.6.5 Noise Barriers and Enclosures

(1) The principal function of noise barriers and noise enclosures is to reduce and shield traffic noise from key sensitive receivers in the vicinity of the roadway or railway. However, the combination of the extent and the substantial structural elements often associated with these facilities demand that aesthetic considerations be applied to reduce the level of adverse visual impacts to the surrounding environmental setting.

(2) Similar to other forms of highway structures, the aesthetic appreciation of noise barriers/enclosures can be made from either the static viewpoint of observers away from the structures, or the dynamic viewpoint of motorists/users along the roadway. From the perspective of static appearance, the noise mitigating structures shall properly fit into the surroundings. Whether the structure shall be integrated into the surroundings or impose a character of its own will depend on the context of the site and its interpretation by the designer. In any case, the overall appearance of the structure shall not adversely affect the visual envelope of the spatial environment. From the motorists' perspective, the noise mitigating structures shall not elicit a feeling of confinement, leading to driver discomfort.

(3) Where noise barriers are installed on road or rail structures, it is vital that the barrier is designed in conjunction with the civil structure and its pattern coordinated with the spacing and dimensions of the structural elements, lighting and other fixtures.

(4) There are many types of noise barrier material available, ranging from concrete, metal, transparent/translucent panels, to composite material panels. During the selection of noise barrier material, consideration shall be given to the principal elements of aesthetic design in terms of the overall appearance of the noise mitigation structure, in addition to the functional performance of the material.

(5) In urban settings, where the close proximity of the built forms usually results in the enlargement of the scale and proportion of the noise mitigation structure relative to the space composition of the surroundings, consideration shall be given to the use of noise panels with properties of transparency and translucency. These properties will visually allow for a feeling of space beyond their surfaces, and will partially conceal traffic from the static observer.

(6) On the other hand, the scale of the noise mitigating structure within the visual envelope of a rural setting is often much smaller due to its relative proportion with the natural elements. This therefore presents opportunities to adopt other types of noise barrier

material in the design of the highway structure. In these instances, the main determination of the noise barrier material to be adopted is likely to be the visual impact, from the perspective of the motorist/user, and the scale and proportion of the barrier relative to the width of the roadway. Vertical elements shall be incorporated to break up the extent of the noise mitigation structure and avoid a sense of monotony being developed by the motorists (this is usually provided in the form of the vertical structural elements supporting the noise barrier panels). However, consideration shall be given to the spacing of the vertical elements so as not to produce significant flicker effects, which may cause driver discomfort.

(7) Similarly, various patterns and features are often incorporated onto the noise barrier panels to create visual and spatial interest for motorist. While this shall not be discouraged, the application of such patterns and features shall be taken with extreme caution. The size, frequency and spacing of these features/patterns will need to be appropriately considered to avoid development of flicker effects, insecurity and associated driver discomfort. For tall, overbearing noise barriers, it may be necessary to incorporate some degree of transparency within the noise mitigation structure to allow the motorist to maintain visual connection with the surrounding environment.

(8) For noise enclosures, an uniformly diffused translucent light-transmitting roof is always preferred, as it will avoid the casting of hard shadows of the enclosure structure on the roadway surface while maximising the use of sunlight during day hours.

15.6.6 Lighting and Signage

(1) The basic function of lighting is to provide for safe use and security, while the primary purpose of signage is to provide directional and other information. Normally, light post and sign post are less attractive elements and the design shall be as less intrusive as possible, especially at visually strategic locations.

(2) Lighting and signage supports shall be minimal, as these supports can clutter the visual environment of the highway structure. The aesthetic goal of these elements is thus to reduce the awareness of the supports. This can be accomplished by grouping of signs, minimising the number of supports or installing these supports where they are not visually disruptive by the use of simple shapes and colour which blends the support into the background.

(3) Road lighting shall not only fulfil its primary function of providing sufficient illumination on roadway/footpath but also enhance the appearance of the highway structure. Too much light creates a disconcerting glare while too little light makes surrounding environments appear gloomy and unsafe. The lighting shall be compatible with its surroundings during the day and help transform the roadway environment into an attractive inviting public place at night.

(4) The material, colour and finish of the light poles, luminaries and other lighting hardware shall be designed to complement the structure’s appearance. Locations of the light poles, where possible, shall be placed with an obvious visual relationship with the piers or other major structural features.

15.7 THE ADVISORY COMMITTEE ON THE APPEARANCE OF BRIDGES AND ASSOCIATED STRUCTURES (ACABAS)

(1) The ACABAS has been set up to enquire into the visual merits, general amenity value and related environmental factors of all proposals to construct bridges and associated structures over, under, on or adjacent to public roads in the Territory.

(2) The designs of all such structures need to be agreed by ACABAS before construction. Submissions to ACABAS shall as far as possible be made at the preliminary design stage. The submission shall include, in general, photos to illustrate site context, future development in vicinity (if the information is available), general layout, elevations and cross sections of the proposed structure to illustrate the scale, proportion and visual relationship among the various elements. Colour/material and surface finish texture samples, as well as details of principal elements that will affect the overall or long term appearance shall also be provided. Experience has shown that photomontages are effective for illustrating the final appearance of a highway structure in relation to its surroundings. Therefore, ACABAS accepts appropriate photomontages as alternatives to perspective views or models required as part of design submissions.

(3) Chromatic design submissions shall contain a summary of the background details, an outline of the design theme adopted and state the principal objectives.

(4) When alterations that will significantly affect the appearance of an existing bridge or associated structure are proposed, details of the proposal should be submitted to ACABAS for agreement prior to implementation. This includes the permanent installation of signs (traffic signs, advertising panels, etc) and utility services.

(5) Reference shall be made to the Environment, Transport and Works Bureau Technical Circular (Works) No. 36/2004 The Advisory Committee on the Appearance of Bridges and Associated Structures (ACABAS), or other latest revision/guideline, on guidance of ACABAS submissions.

(6) Other public consultation such as submission to District Council or its sub-committees shall also be made if considered necessary. Views collated from the public shall thoroughly be examined and considered during the course of design works. The designer shall initiate another consultation cycle if found any subsequent alteration on the structure that will likely affect the appearance of a structure or deviate from the original design theme. Priority shall always be given to the commitment on the provision of competent design acceptable to the general public and the like.

Figure 15.1 – Aesthetic Concepts and Considerations

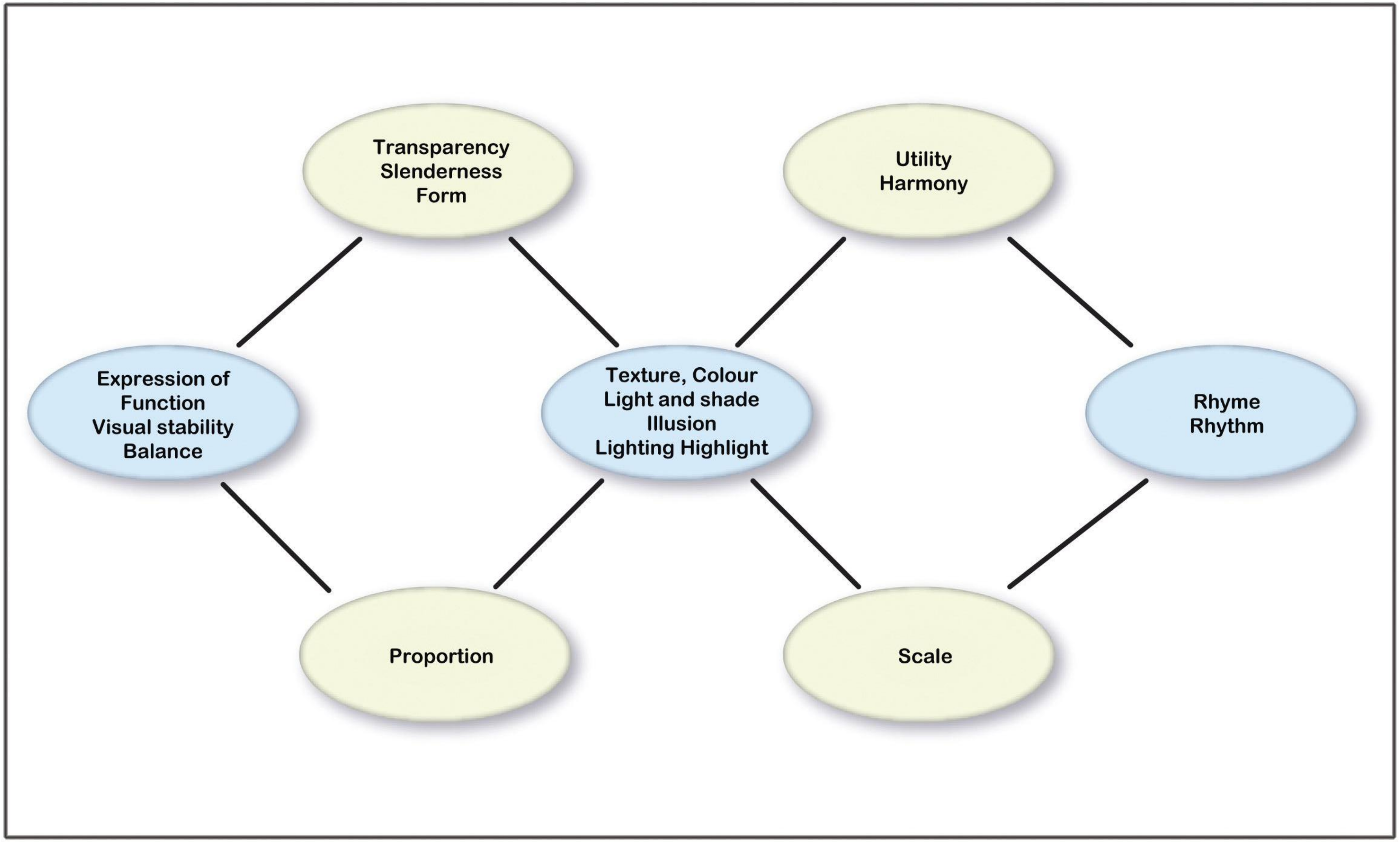
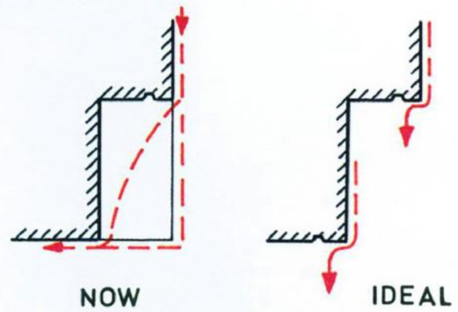


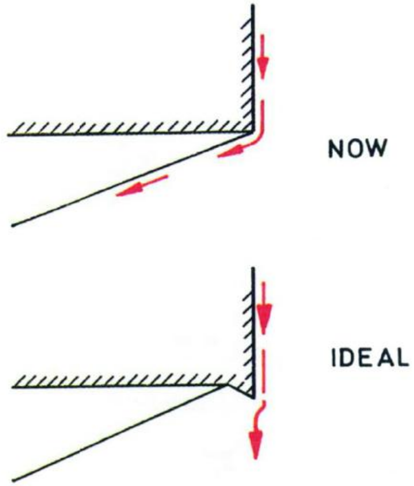
Figure 15.2 – Long Term Appearance : Wash-water Staining (Sheet 1 of 3)



A: • drip-lines interrupted at higher level

• no drip-lines at soffit

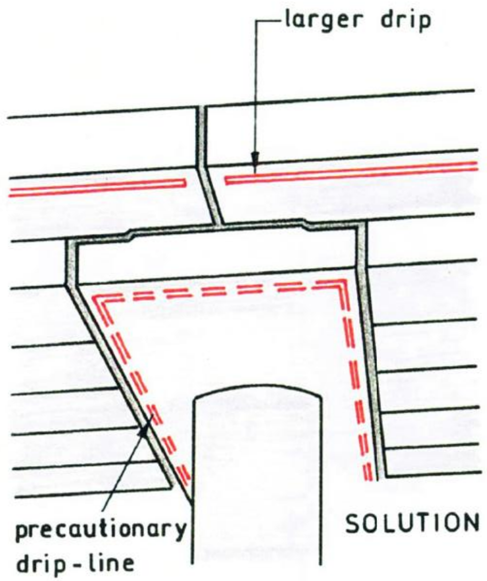
Figure 15.3 – Long Term Appearance : Wash-water Staining (Sheet 2 of 3)



B: • support hammer-heads with no drip forms

• leak joints

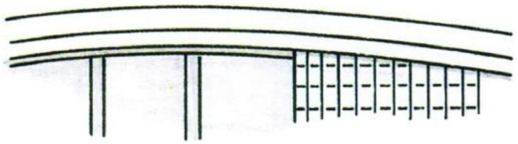
Figure 15.4 – Long Term Appearance : Wash-water Staining (Sheet 3 of 3)



C: • drip groove too small

• leaky joints

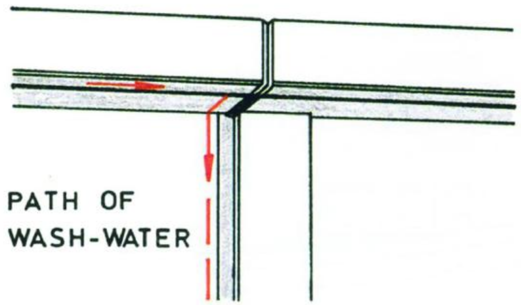
Figure 15.5 – Long Term Appearance (Sheet 1 of 3)



A: • note uniform depth and good continuity of parapet line from bridge, on along top of abutment wall, enhanced by shadow

• effective planting helps to 'soften' hard structure

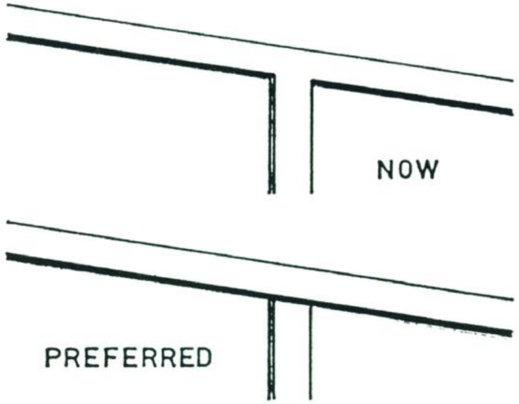
Figure 15.6 – Long Term Appearance (Sheet 2 of 3)



B: • good joint/pier/drip junction to minimize risk of staining

• good line of parapet, not interrupted by shallow pier

Figure 15.7 – Long Term Appearance (Sheet 3 of 3)



C: • vertical pier interferes with line of parapet

• risk of staining on pier due to interruption of drip

Figure 15.8 – Transparency and Slenderness

Figure 15.9 – Structural Form (Sheet 1 of 3)

(a) Cable-stayed

(b) Suspension

Figure 15.10 – Structural Form (Sheet 2 of 3)

(c) Simple beam bridge

(d) Arch bridge

Figure 15.11 – Structural Form (Sheet 3 of 3)

(e) Trussed structural steel

Figure 15.12 – Dimensions Governing Proportion

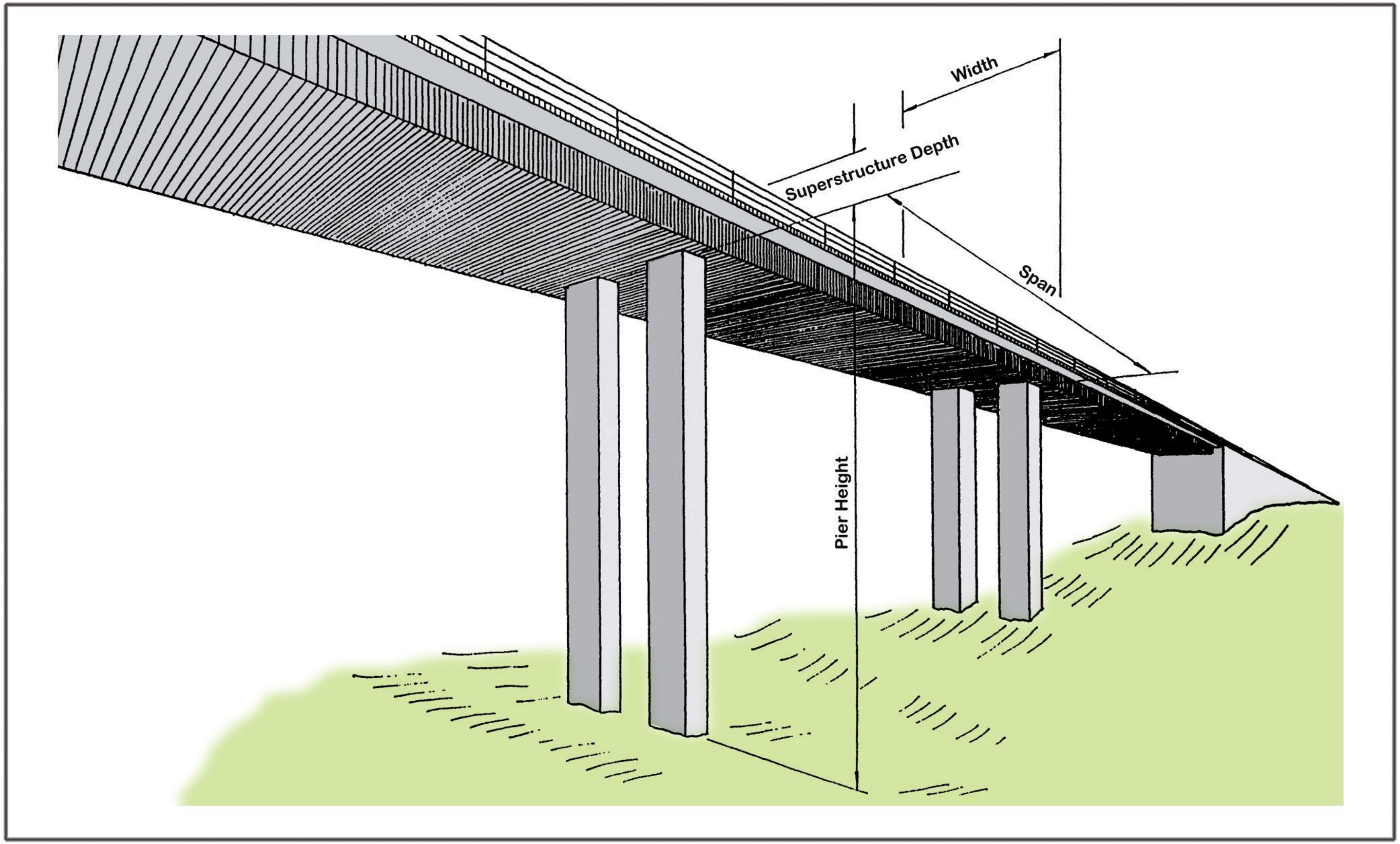


Figure 15.13 – Failure of Two-Dimensional Drawing to Indicate Bulkiness

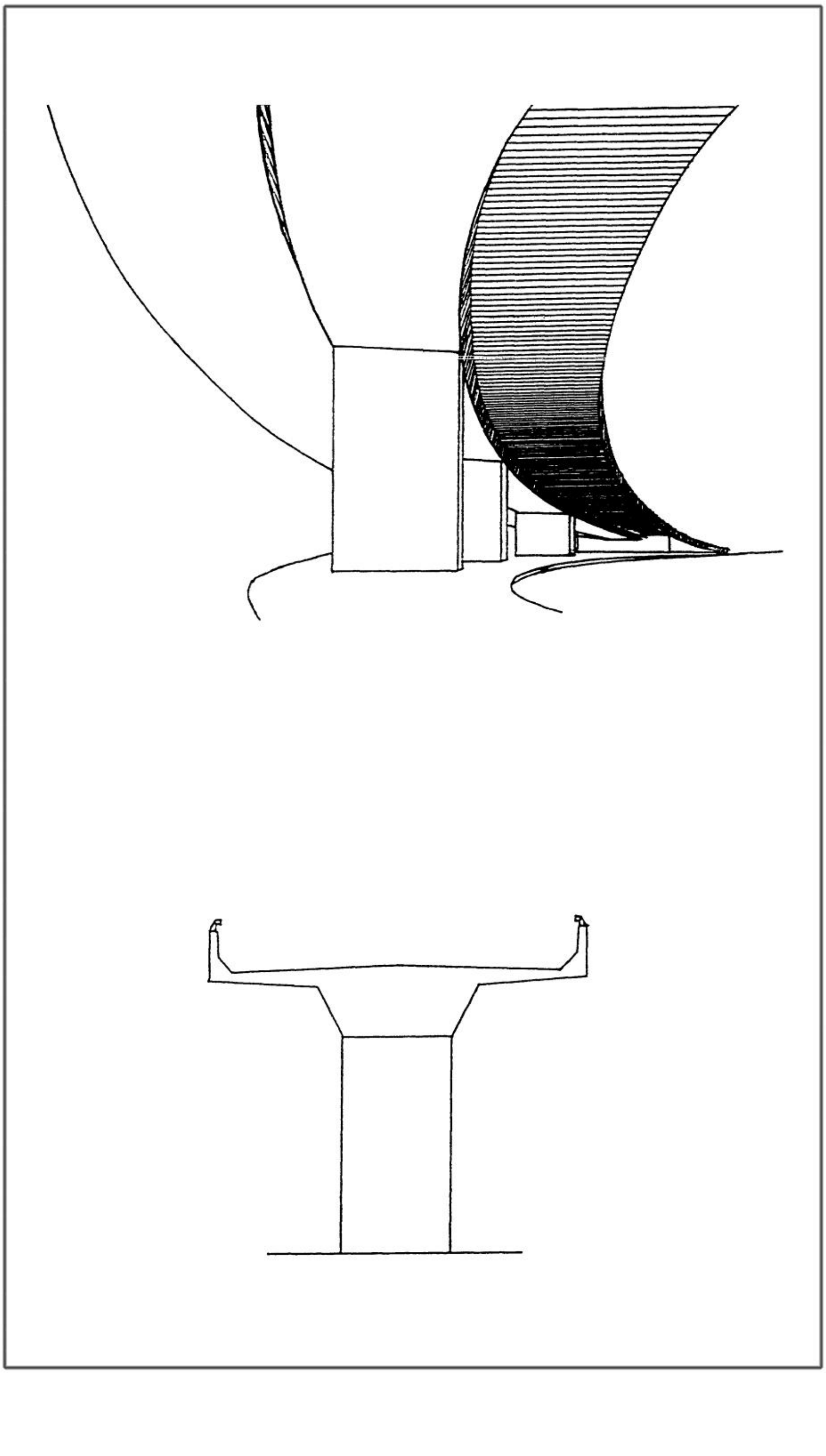


Figure 15.14 – Texture, Pattern and Scale

A: The wall shown above is smaller than that shown in the picture B below, but the pattern of panelling is large, increasing its apparent scale.

B: Large wall but with small scale texture is less obtrusive than if it were divided into large panels.

Figure 15.15 – Expression of Function – Smoothness of Flow

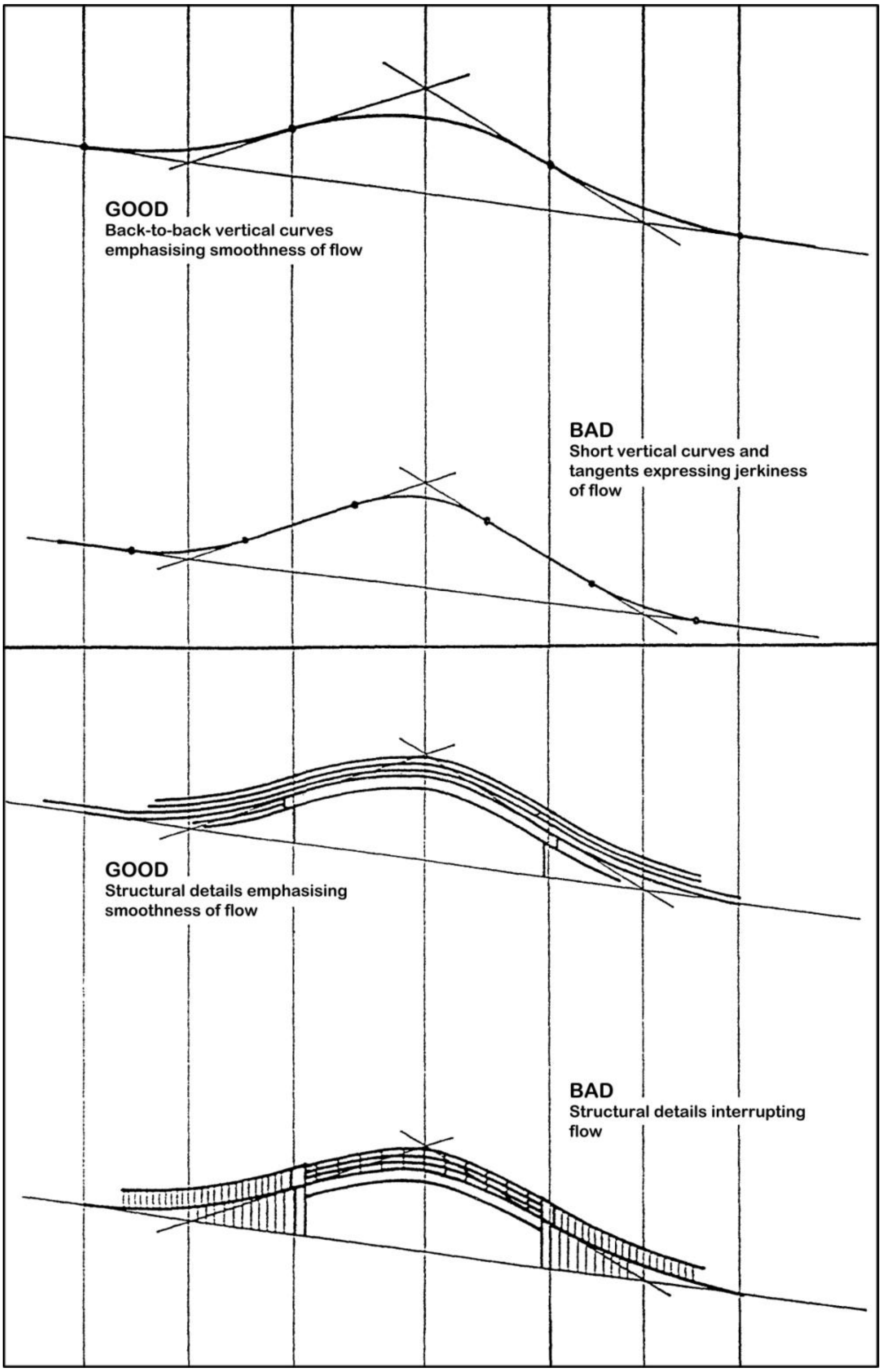


Figure 15.16 – Vandalized Form – Poor Expression of Function

Curvature is one of the bridge designer's best allies. To destroy curvature for the sake of easy design is to seriously compromise the achievement.

Figure 15.17 – Expression of Function

Place of descent

The tilted wall indicates this is a place to descend

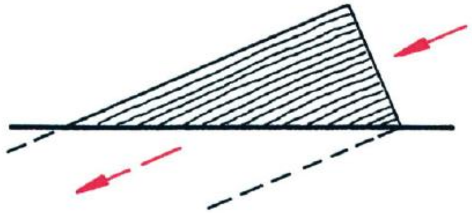


Figure 15.18 – Expression of Function – Stability

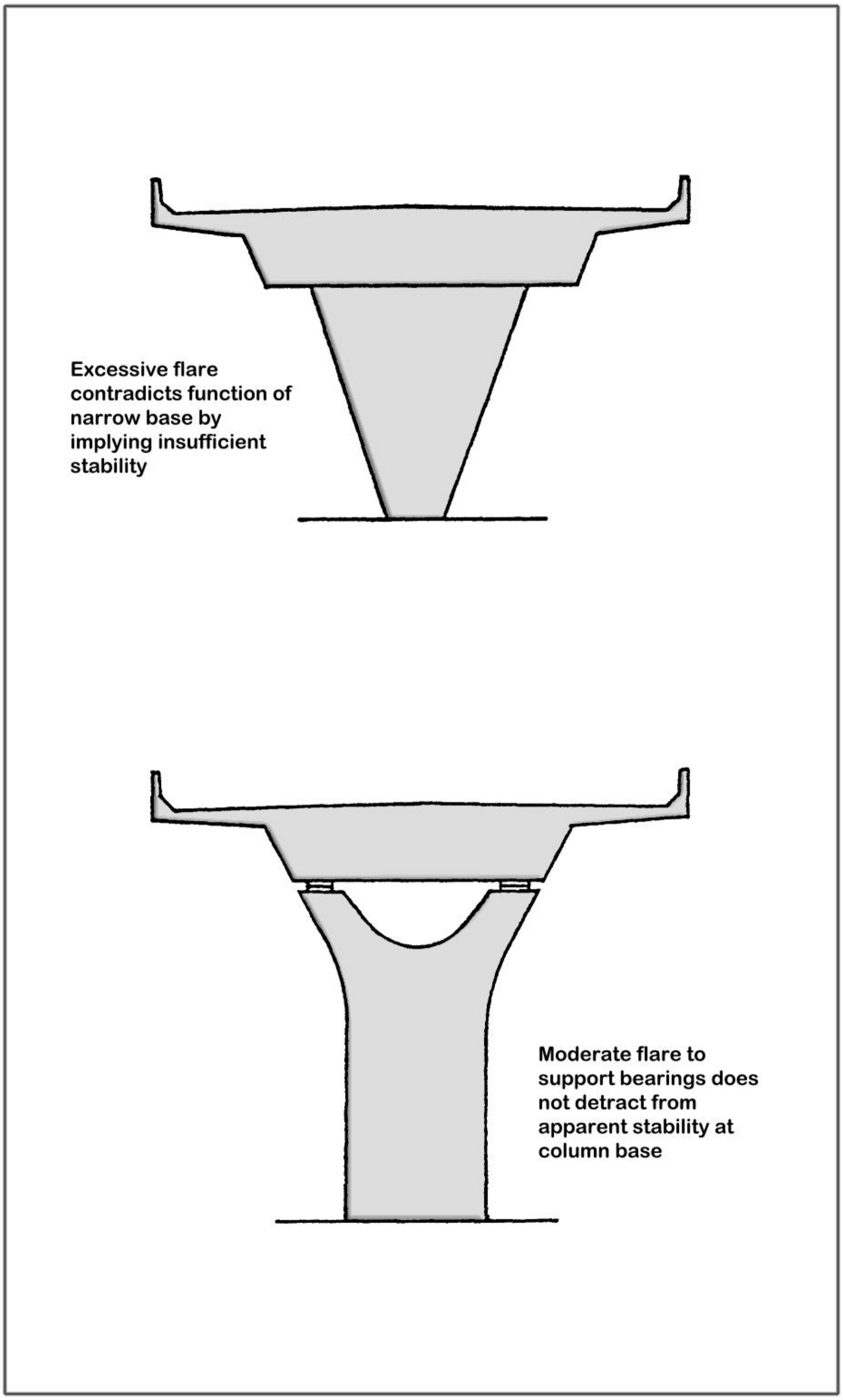


Figure 15.19 – Unity and Harmony

The form, rhythm, colour and finishing of this footbridge blends in a positive way with the adjacent landmark commercial building resulting in unity and harmony of the bridge with the surrounding landscape and townscape.

Figure 15.20 – Visual Instability Arising from the Use of Trapezoidal Support

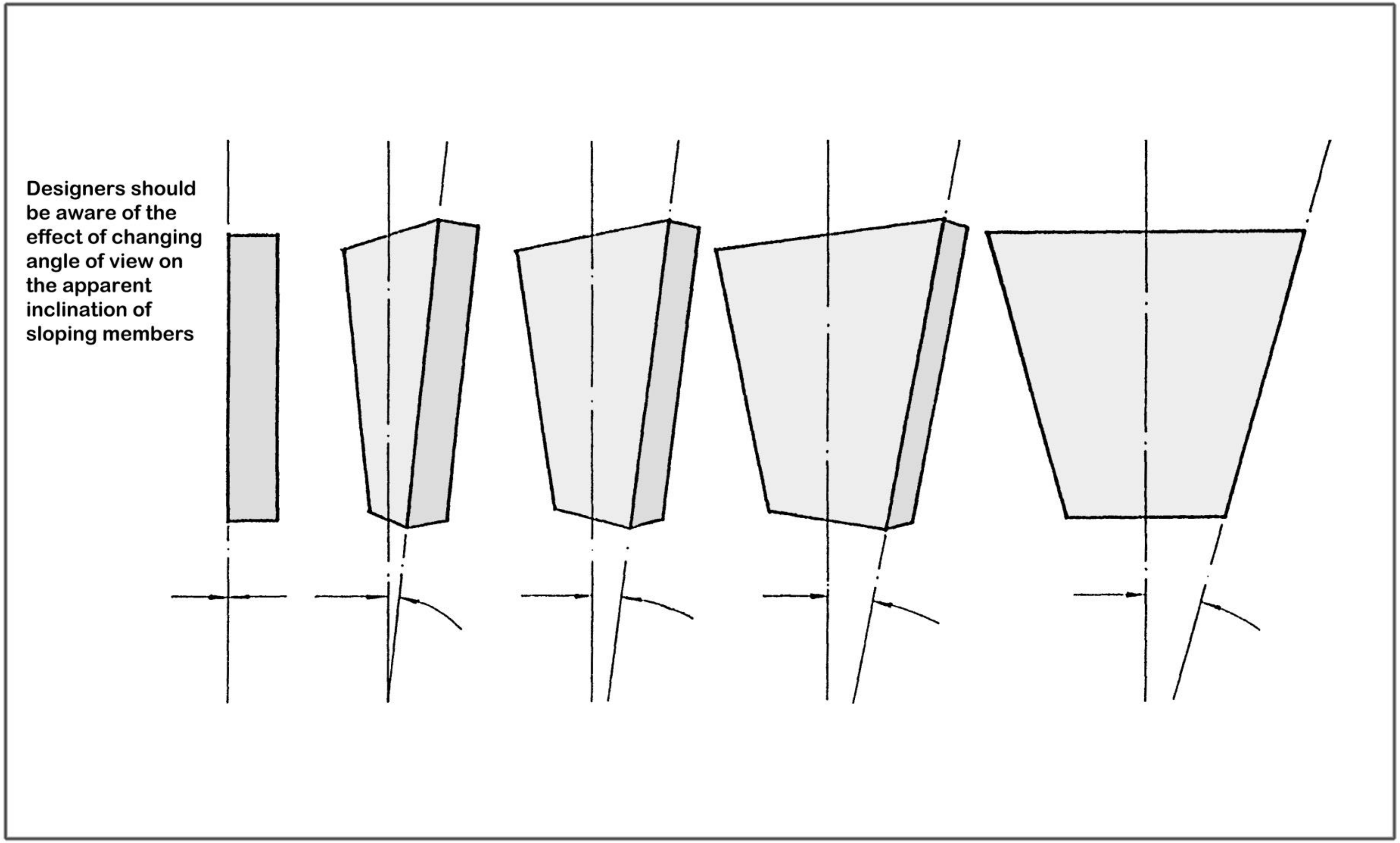
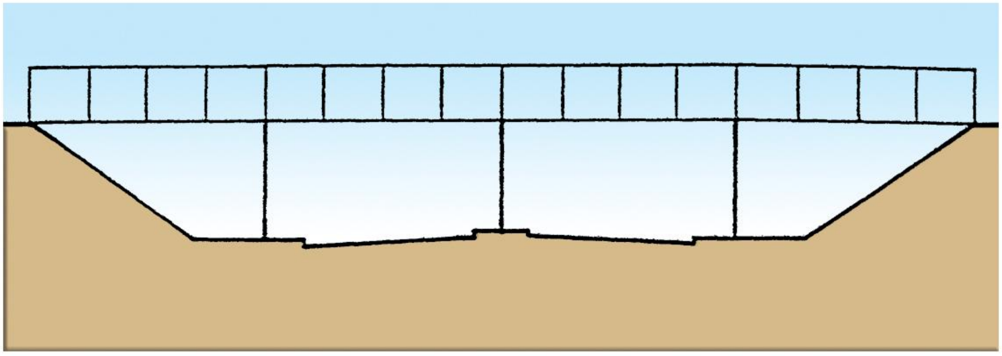
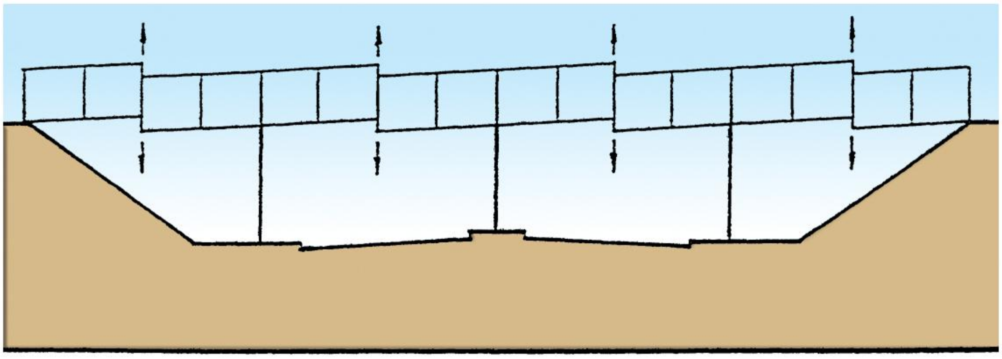
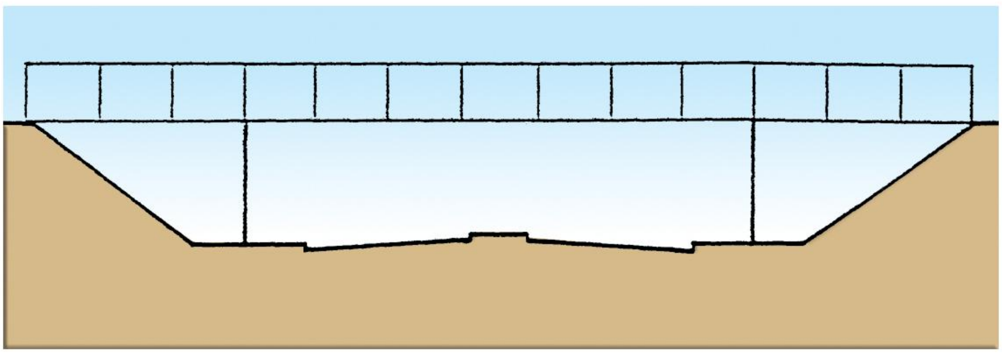


Figure 15.21 – Visual Instability due to Unresolved Duality





BAD Even number of modules creates visual instability



GOOD Odd number of modules creates balance

Figure 15.22 – Illusion of Sag due to Central Support

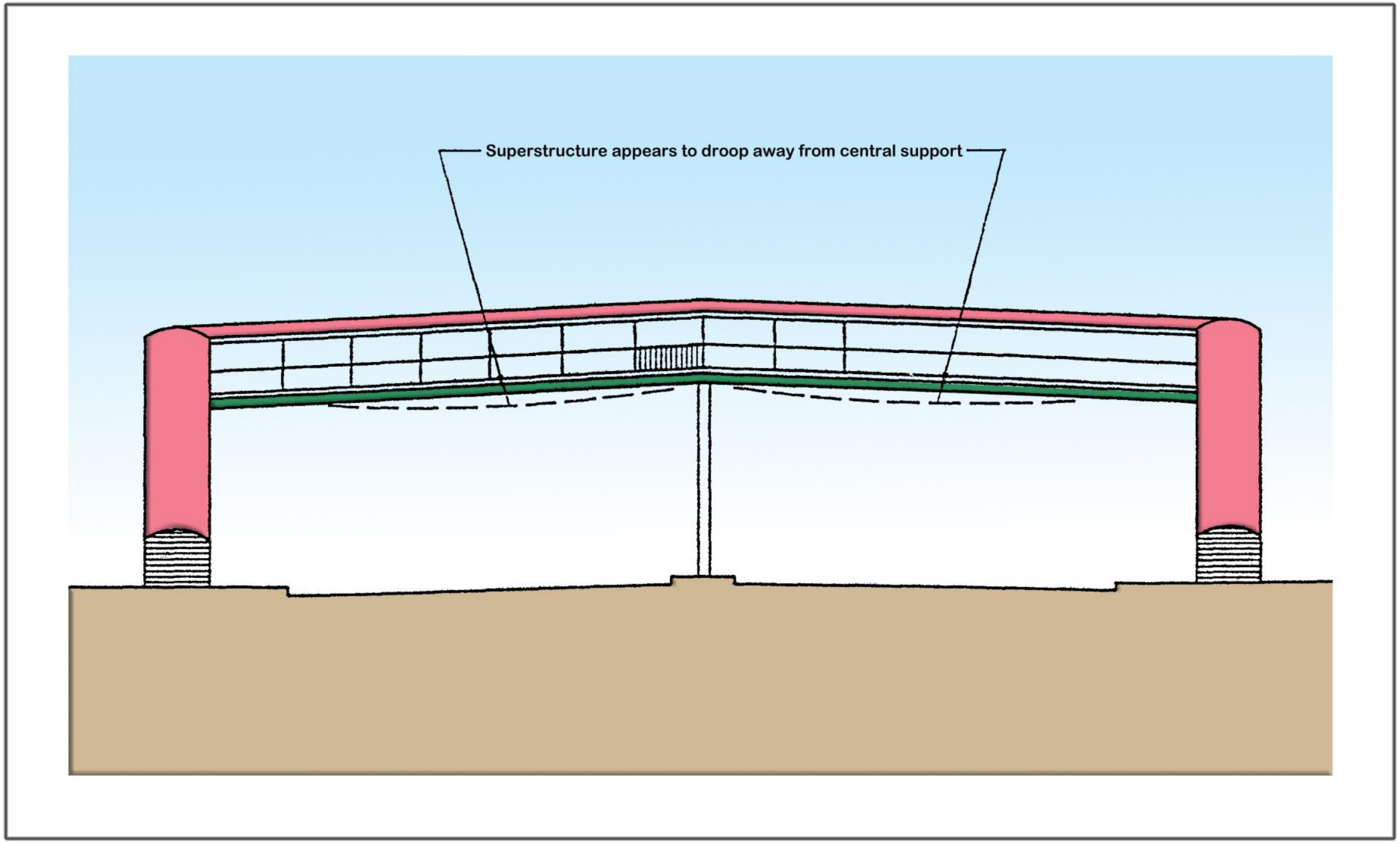


Figure 15.23 – Rhythm

(a) Ap Lei Chau Bridge : Poor Rhythm

(b) Footbridge in Sha Tin : Good Rhythm

Figure 15.24 – Rhythm and Rhyme

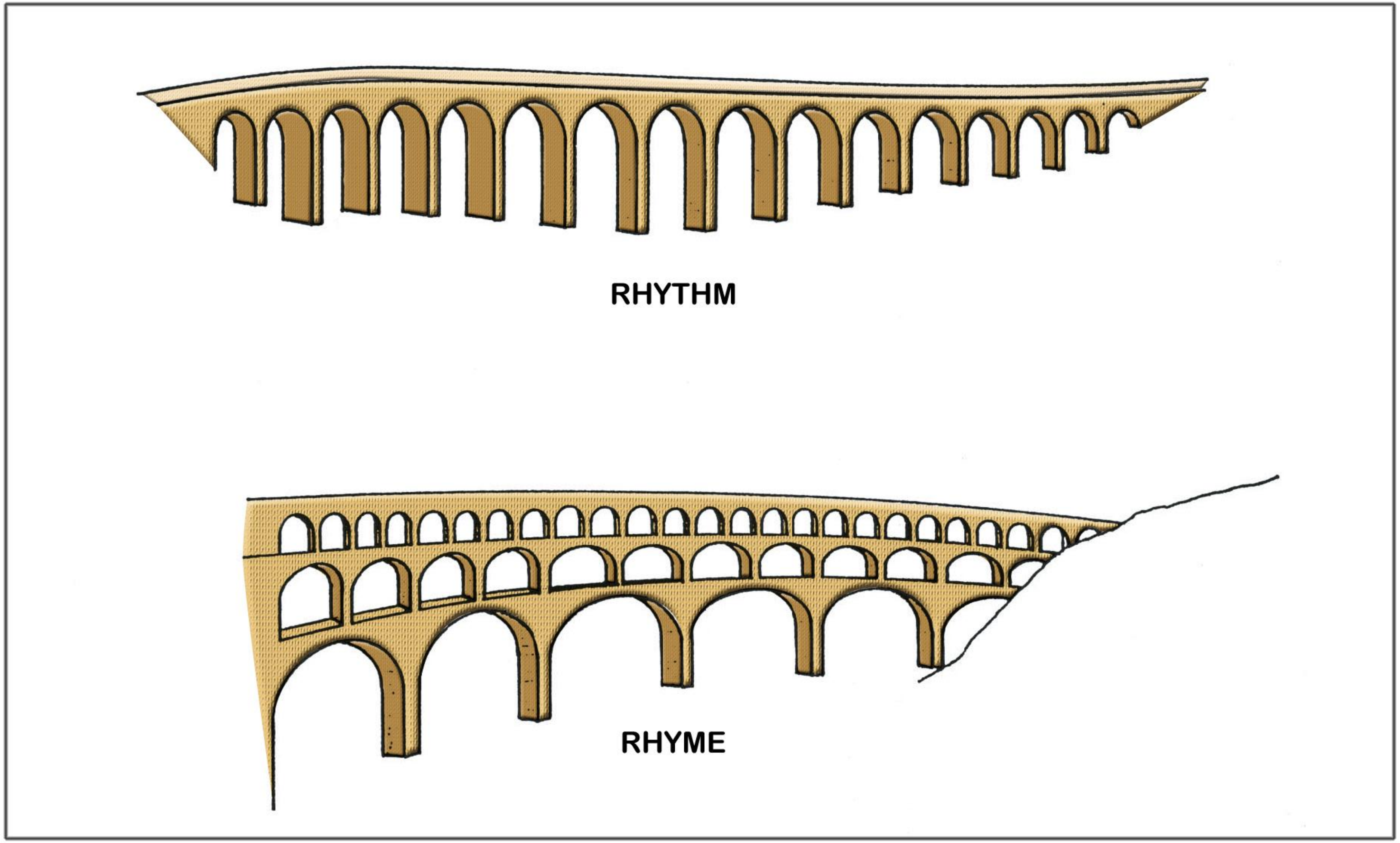


Figure 15.25 – Light and Shade

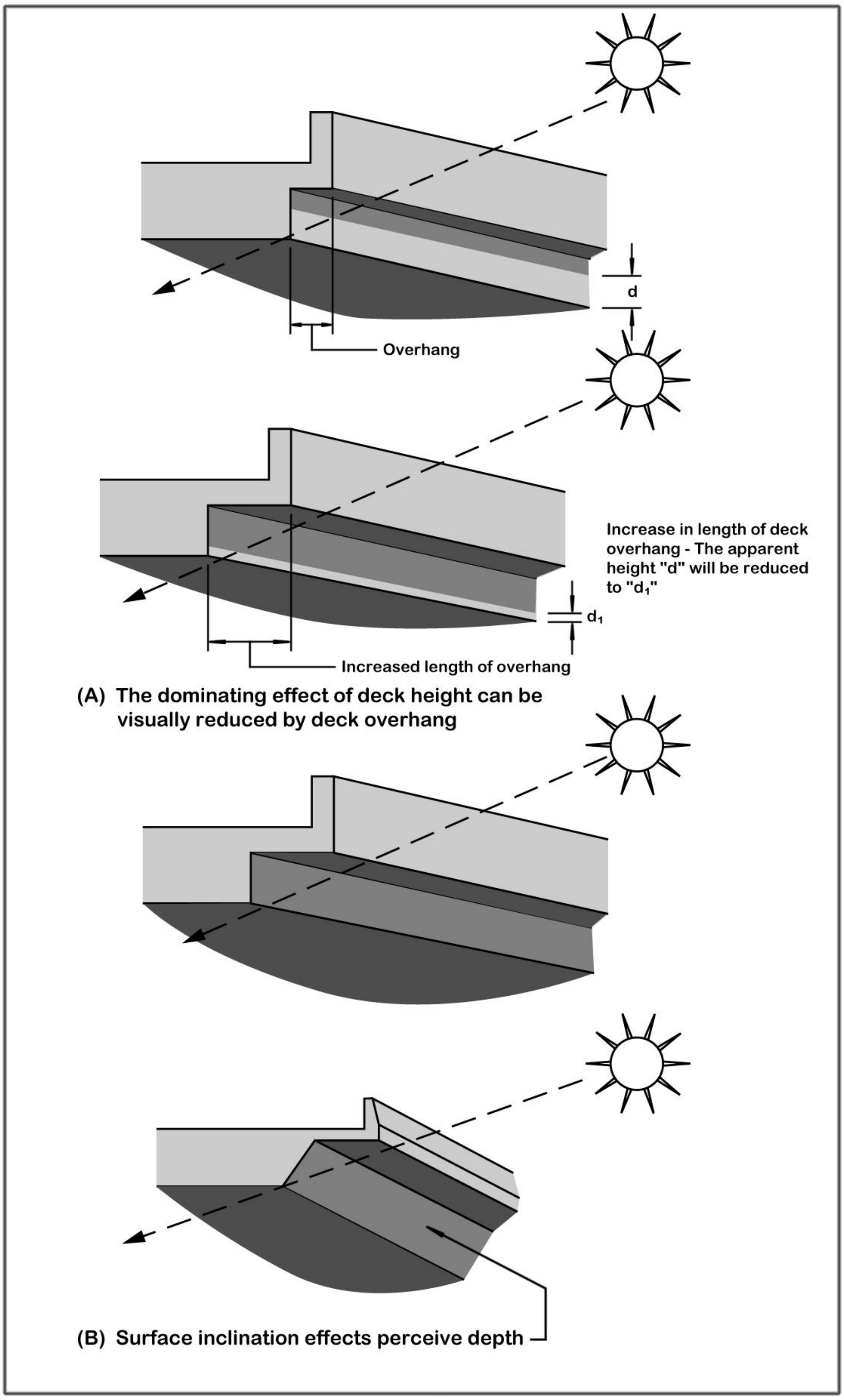


Figure 15.26 – Formed and Applied Texture (Sheet 1 of 2)

Various textures add visual interest, can help to define different functional elements such as smooth deck, rugged abutment shown here

Figure 15.27 – Formed and Applied Texture (Sheet 2 of 2)

Combination of paint and steel cladding finish to canopy soffit

Figure 15.28 – Chromatic Design

Attention to detail and highlighting elements to express function, direction or to create interest are all essential aspects of comprehensive design

Figure 15.29 – Lighting Highlight

(a) Lighting arrangement of Ting Kau Bridge

(b) Lighting arrangement of Tsing Ma Bridge

CHAPTER 16 OPERATIONAL CONSIDERATIONS

16.1 GENERAL CONSIDERATIONS

(1) In the design of highway structures, due consideration should be given to durability during the service life. The materials and structures shall resist for the target period and with regular maintenance, all the effects to which they are subjected, so that no significant change occurs in their serviceability. Achievement of durability is primarily affected by design and detailing, material specifications and quality of construction. The cost of maintenance should be considered besides the cost of capital construction.

(2) The type of structure selected for a particular location and the working conditions it is subjected to can have an important bearing on its durability. The specific durability requirements of a structure should be assessed during the design stage and measures for their achievement should be considered. Such considerations may include, but not limited to, the followings :

(a) Provision of adequate cover to reinforcement (see Clause 5.4.2).

(b) The use of corrosive protection measures such as waterproofing membranes, epoxy coated reinforcement, cathodic protection, etc. The designer is responsible for selecting the corrosion protection system most appropriate to the structure. The designer should take into account technology contemporary at the time the design is prepared, the types and properties of corrosion protection systems available, and the drainage characteristics of the structure. In the case of a bridge deck, the effects of the provision of the corrosion system on the running surface should also be considered. Reference can be made to Technical Report No. RD/TR/039 “Corrosion Protection of Concrete Bridge Decks” and Guidance Notes No. RD/GN/033 “Guidance Notes on the Use of Waterproofing Membranes on Concrete Bridge Decks” published by the Highways Department.

(c) Provision of proper and adequate access for inspection and maintenance (see Clause 16.1.1)

(d) Provision of a positive, well designed, detailed and constructed drainage system for managing water from the structure, and into a drainage system (see Chapter 14).

16.1.1 Access for Inspection and Maintenance

(1) Highway structures and railway bridges require regular inspection and maintenance in the course of their service life. Consideration shall accordingly be given at the design stage to the provision of means of safe access to all locations and components for inspection and maintenance. Such means of access may include step-irons, ladders, cat-walks, abutment chambers, gantries and inspection openings and covers as appropriate, and shall be designed to prevent public from misuse of any of these access facilities and colonisation of the areas in question by plants, animals or birds. These access facilities

shall be provided at all abutments and columns wherever practical having due regard to the appearance and functions of the structure. Railway authorities shall also be consulted if the access facilities are to be provided at a level above the nearby railway tracks.

(2) In determining the location of access points, it should preferably be at each end of the structure at points which are safe and easily accessible and do not require traffic control. Access shall be provided from below deck level, to avoid access through deck surfaces.

(3) A minimum of one inspection opening not less than 600 x 600 mm or 700 mm in diameter shall be provided as external access into every span of cellular structures with internal vertical dimension greater than 1200 mm. If such an arrangement is considered impractical, prior consent shall be sought from the maintenance authority for any deviation from the above requirement. Watertight covers which do not rely solely on greasing for achieving watertightness shall be used for inspection openings through the top surface of the structure. Hinged type cover fabricated from corrosion resistant material and compatible with the surrounding concrete finishes shall be used for inspection openings through the underside of the structure. Inspection opening not less than 600 x 600 mm or 700 mm in diameter shall be provided in longitudinal webs in multi-cell structures. If possible, additional access openings not less than 800 mm wide by 1000 mm high without doors should also be provided through the internal diaphragms. Adequate ventilation and drainage holes shall be provided to all closed cells or box sections. Consideration shall be given to the provision of adequate artificial lighting inside cells with length more than 60 m long.

(4) Post-tensioned structures using external or unbonded tendons should be detailed such that inspection of all individual tendons is possible without restriction on highway traffic.

16.1.2 Maintenance Accommodation

(1) Maintenance is easier to carry out if workshops and stores are available close at hand. Consideration shall therefore be given at the design stage to the provision of suitable workshop and store accommodation. Most major structures have approaches in which the necessary accommodation can be provided easily. The maintenance authority shall be consulted at an early stage in the design to establish whether there is a need for such accommodation.

(2) The workshop shall have a sink with water supply, a toilet and electricity supply for lighting and power. The store shall have lighting and shall be separated from the workshop by a lockable door. Access to the workshop shall be by means of a 2000 mm by 2000 mm opening with stout galvanized steel double doors secured with bolts and padlocks. Workshops and stores shall be suitably ventilated.

(3) If possible, a parking bay shall be provided for use by maintenance vehicles. Precautions shall be taken against unauthorized use of such parking bays.

16.1.3 Spare Parts

(1) Certain components are susceptible to damage or wear during the service life of a highway structure or railway bridge. A range of spare parts shall be provided for such components upon their handing over to the maintenance authority. Certain components such as tiles, cladding panels and special parapet railings are susceptible to damage or wear during the service life of a highway structure or railway bridge. A range of spare parts shall be provided for such components upon their handing over to the maintenance authority. The aim of this provision is to achieve economy in overall cost and to ensure proper maintenance. In general, for those components which (or equivalent) are readily available in local market, it is not cost effective to keep any more stocks. The maintenance authority shall be consulted at an early stage in the design of the structures for advice on the requirements for such.

(2) The project proponents, when requested by the maintenance authority, shall provide accommodation within the new structures to store the spare parts.

16.2 SAFETY CIRCUITS FOR BRIDGES OVER NAVIGABLE CHANNELS

(1) Ships often collide with bridges over navigable channels. The severity of damage caused by such collisions depends on the design of the bridges, but fatalities can result from vehicles falling into waterways because their drivers are unaware of collision damage.

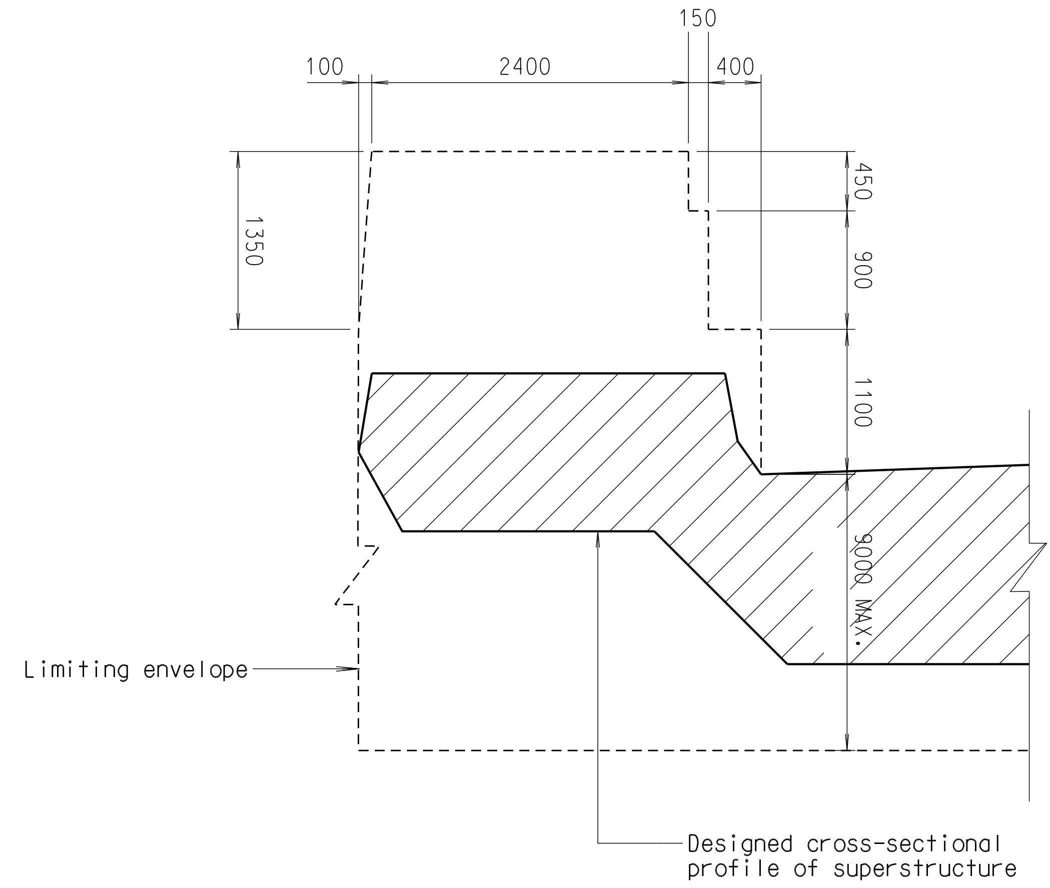
(2) Bridges exposed to ship collisions shall be provided with safety circuits which activate stop lights to halt passing traffic if the bridge is seriously damaged.

(3) Such bridges are also subjected to less severe blows from passing vessels which may not be severe enough to cause collapse but can result in a damage that may not be readily apparent and needs repairs. Secondary safety circuits capable of detecting and recording such blows shall be fitted to draw attention to the need for inspection and maintenance.

16.3 LIMITING ENVELOPE FOR STRUCTURAL ELEMENTS AND INSTALLATIONS

(1) To facilitate the use of the underbridge inspection vehicle for the inspection and maintenance of the underside of the structure, no part of the structure or any projection therefrom, except road lighting, fire hydrants, emergency telephones and noise barrier, shall protrude beyond the limiting envelope as defined by the shaded area in Figure Figure 16.1.

Figure 16.1 – Limiting Envelop for Highway Structure



(2) The prior agreement of the maintenance authority shall be obtained at an early stage in the design of the structure for incorporating any continuous or isolated installations which protrude beyond this limiting envelope. Nevertheless any such continuous installations shall be easily removable and the clear distance between the isolated protruding elements which are not easily removable shall not be less than 2000 mm.

16.4 PAINTING OF STEELWORK

(1) All steelwork, including sign gantries and vehicle and pedestrian parapets, shall be either hot dip galvanized or metal sprayed and shall in addition be painted. As in all painting work, the surface to be painted shall be thoroughly cleaned and prepared, free from grease, dirt, scale and rust in accordance with the requirements of BS EN ISO 12944 Part 4.

(2) The volatile organic compounds (VOC) content for the paints shall be in compliance with the Air Pollution Control (Volatile Organic Compounds) Regulation (the Regulation), and shall not exceed the maximum limits of VOC content for the Regulated Architectural Paints as listed in the Regulation. As a general reference for highway works, paints classified as "Industrial Maintenance Coatings" with a VOC content limit of 250g/L under the Regulation are appropriate paint materials to be used for compliance.

(3) Direct application of paint to newly galvanized steelwork will result in premature failure of the paint system. Such failures are usually due to the formation of brittle zinc soaps at the paint/zinc interface with the resultant loss of adhesion and deterioration in the properties of the paint film. The pretreatment of the surface with a proprietary two pack etch primer prior to painting would prevent the failure of the paint system. Primers shall be applied in thin coats by continuous spraying and strictly in accordance with the manufacturer's instructions. Suitable one pack primers are also available, but care must be taken to ensure that they are formulated for use on galvanized steel.

(4) Weathering of galvanized surfaces until all bright zinc has changed to a dull surface by oxidation may aid adhesion of the paint, provided any loose particles have been removed from the surface. The deliberate use of weathering as a pretreatment for painting is not recommended as the minimum time needed for full weathering cannot easily be assessed. It may also be difficult to completely clean a weathered surface in preparation for painting.

(5) After galvanized or metal sprayed structures have been painted, subsequent maintenance will be of the paint system. The paint systems, and their required life to first maintenance of the paint system in very high marine corrosivity (C5-M) environment as defined in BS EN ISO 12944 Part 5, to be used for painting galvanized or metal sprayed steelworks shall be :

(a) *Paint System I*

To be applied to : parapets, etc.

Life to first maintenance : 5 to 15 years, medium durability as defined in BS EN ISO 12944 Part 5

Pretreatment : two-pack etch primer

Primer : two-pack epoxy primer, 80 μm minimum total dry-film thickness

Finish : two pack epoxy finish coat or polyurethane finish coat, 80 μm minimum total dry-film thickness

(b) *Paint System II*

To be applied to : structural steelworks

Life to first maintenance : more than 15 years, high durability as defined in BS EN ISO 12944 Part 5

Pretreatment : two-pack etch primer

Primer : two-pack epoxy zinc phosphate primer, 80 μm minimum total dry-film thickness

Undercoat : two-pack micaceous iron oxide epoxy undercoat, 140 μm minimum total dry-film thickness

Finish : two-pack polyurethane finish coat, 100 μm minimum total dry-film thickness

(c) *Paint System III*

To be applied to : metal sprayed surfaces

Lift to first maintenance : more than 15 years, high durability as defined in BS EN 1SO 12944 Part 5

Pretreatment : two-pack zinc tetroxychromate polyvinyl butyral pretreatment

Sealer : two-pack epoxy sealer applied by brush until absorption is complete

Primer : two-pack epoxy zinc phosphate primer, 80 μm minimum total dry-film thickness

Undercoat : two-pack micaceous iron oxide epoxy undercoat, 140 μm total minimum dry-film thickness

Finish : two-pack polyurethane finish coat, 100 μm minimum total dry-film thickness

(6) The aforesaid guidelines shall not be applicable to exceptionally massive steelwork, such as the steel deck of the Tsing Ma Bridge, Ting Kau Bridge, etc., where special corrosive protection system shall be considered with regard to the particular project requirements.

16.5 INCORPORATION OF UTILITY INSTALLATIONS IN HIGHWAY STRUCTURES

(1) In general no utility installations other than road lighting, emergency telephones and traffic surveillance equipment will be permitted on highway structures except in cases where there is no other viable routing available. Where other arrangements for a utility line to span an obstruction are not viable nor practical, the Highways Department may consider the accommodation of such line in a highway structure if the proposed

accommodation will not materially affect the structure, the safe operation of traffic, the efficiency of maintenance and the overall appearance.

(2) The prior approval of Chief Highway Engineer/Bridges and Structures and the maintenance authority shall be sought on any proposal to accommodate utility installations other than road lighting, emergency telephones and traffic surveillance equipment in highway structures. The need for accommodating utility installations should be confirmed at an early stage in the design to allow the designer to make adequate and appropriate provision having due regard to the appearance and functions of the structure. The following guides are established for making provision for accommodation of utilities in highway structures :

(a) The utility lines or installations shall be accommodated in a purpose built trough accessible from the footway or verge, rather than fixed to the sides or underside of the structure using steel brackets. Funding for any additional costs for the provision of the trough is outside the ambit of this Manual and will be dealt with separately.

(b) Encasing utility installations inside the structural elements of the structure including any internal voids is not permitted.

(c) No gas main shall be accommodated in a highway structure which carries a strategic route and a sole access and if there are serious consequences in case the structure is damaged by possible gas explosions.

(d) The covers, or covers and frames, for the troughs shall fit closely together to prevent lateral movement or rocking of the covers under traffic. The gap between covers, or covers and frames, shall not exceed 3 mm when assembled in continuous lengths.

(e) Where possible the space under footways and verges should be designed to permit the installation of small utilities at a later date.

16.6 MATERIALS FOR HOLDING DOWN AND FIXING ARRANGEMENTS ON HIGHWAY STRUCTURES

(1) The holding down and fixing arrangements of all sign gantries, noise barriers and the like, and all other fixtures on highways structures shall be fabricated from austenitic stainless steel. In detailing the holding down and fixing arrangements, necessary measures must be provided to prevent galvanic corrosion arising from bi-metallic contact. Stainless steel materials shall comply with Section 18 of the General Specification for Civil Engineering Works, except that Grade 1.4401 shall be replaced by Grade 1.4436 and stainless steel tube shall be Grade 1.4436.

(2) Galvanized mild steel fixing arrangements may only be used for internal fixtures.

16.7 RUNNING SURFACES OF BRIDGE DECKS

(1) To achieve better riding quality and to allow greater flexibility in maintaining the running surfaces of highway bridge decks, the bridge deck surface shall be designed to be finished with bituminous materials in accordance with Highways Department Technical Circular No. 11/2001 "Running Surfaces of Bridge Decks". Due consideration shall also be given to the Guidance Notes No. RD/GN/033 "Guidance Notes on the Use of Waterproofing Membranes on Concrete Bridge Decks" published by the Highways Department in designing the bituminous surfacing.

(2) A concrete running surface shall only be considered for sections where a short structure is located within a length of rigid carriageway.

APPENDIX A

LIST OF EUROCODES, UK NATIONAL ANNEXES AND PUBLISHED DOCUMENTS

CONTENTS

A.1 INTRODUCTION

A full list of Eurocodes, UK National Annexes (UK NA) and Published Documents (PDs), which form the basis of this Manual for the design of highway structures and railway bridges in Hong Kong is provided in Clauses A.2 to A.4.

A.2 EUROCODES

<u>Eurocode : Basis of Structural Design</u>	BS EN 1990:2002+A1:2005 *Incorporating Corrigenda December 2009 and April 2010*
<u>Eurocode 1: Actions on Structures –</u>	
Part 1-1: General actions – Densities, self-weight, imposed loads for buildings	BS EN 1991-1-1:2002 *Incorporating Corrigenda December 2004 and March 2009*
Part 1-4: General actions – Wind actions	BS EN 1991-1-4:2005+A1:2012 *Incorporating Corrigenda July 2009 and January 2012*
Part 1-5: General actions – Thermal actions	BS EN 1991-1-5:2003 *Incorporating Corrigenda December 2004 and March 2009*
Part 1-6: General actions – Actions during execution	BS EN 1991-1-6:2005 *Incorporating Corrigendum July 2008*
Part 1-7: General actions – Accidental actions	BS EN 1991-1-7:2006 *Incorporating Corrigendum February 2010*
Part 2: Traffic loads on bridges	BS EN 1991-2:2003 *Incorporating Corrigenda December 2004 and February 2012*
<u>Eurocode 2: Design of Concrete Structures –</u>	
Part 1-1: General rules and rules for buildings	BS EN 1992-1-1:2004 *Incorporating Corrigenda January 2008 and November 2010*
Part 2: Concrete bridges – Design and detailing rules	BS EN 1992-2:2005 *Incorporating Corrigendum July 2008*

Eurocode 3: Design of Steel Structures –

Part 1-1: General rules and rules for buildings	BS EN 1993-1-1:2005 *Incorporating Corrigenda February 2006 and April 2009*
Part 1-5: Plated structural elements	BS EN 1993-1-5:2006 *Incorporating Corrigendum April 2009*
Part 1-6: Strength and stability of shell structures	BS EN 1993-1-6:2007 *Incorporating Corrigendum April 2009*
Part 1-7: Plate structures subject to out of plane loading	BS EN 1993-1-7:2007 *Incorporating Corrigendum April 2009*
Part 1-8: Design of joints	BS EN 1993-1-8:2005 *Incorporating Corrigenda December 2005, September 2006, July 2009 and August 2010*
Part 1-9: Fatigue	BS EN 1993-1-9:2005 *Incorporating corrigenda December 2005, September 2006 and April 2009*
Part 1-10: Material toughness and through-thickness properties	BS EN 1993-1-10:2005 *Incorporating Corrigenda December 2005, September 2006 and March 2009*
Part 1-11: Design of structures with tension components	BS EN 1993-1-11:2006 *Incorporating Corrigendum April 2009*
Part 2: Steel bridges	BS EN 1993-2:2006 *Incorporating Corrigendum July 2009*

Eurocode 4: Design of Composite Steel and Concrete Structures –

Part 1-1: General rules and rules for buildings	BS EN 1994-1-1:2004 *Incorporating Corrigendum April 2009*

Part 2: General rules and rules for bridges	BS EN 1994-2:2005 *Incorporating Corrigendum July 2008*

Eurocode 8: Design of Structures for Earthquake Resistance –

Part 1: General rules, seismic actions and rules for buildings	BS EN 1998-1:2004 *Incorporating corrigendum July 2009 and January 2011*
Part 2: Bridges	BS EN 1998-2:2005+A2:2011 *Incorporating Corrigenda February 2010 and February 2012*

A.3 UK NATIONAL ANNEXES

UK National Annex for Eurocode : Basis of Structural Design	UK NA to BS EN 1990:2002 +A1:2005 *Incorporating National Amendment No.1*

UK National Annex to Eurocode 1: Actions on Structures –

Part 1-1: General actions – Densities, self-weight, imposed loads for buildings	UK NA to BS EN 1991-1-1:2002
Part 1-4: General actions – Wind actions	UK NA to BS EN 1991-1-4:2005+A1:2012 *Incorporating National Amendment No.1*
Part 1-5: General actions – Thermal actions	UK NA to BS EN 1991-1-5:2003
Part 1-6: General actions – Actions during execution	UK NA to BS EN 1991-1-6:2005
Part 1-7: General actions – Accidental actions	UK NA to BS EN 1991-1-7:2006
Part 2: Traffic loads on bridges	UK NA to BS EN 1991-2:2003 *Incorporating Corrigendum No.1*

UK National Annex to Eurocode 2: Design of Concrete Structures –

Part 1-1: General rules and rules for buildings	UK NA to BS EN 1992-1-1:2004 *Incorporating National Amendment No.1*

Part 2: Concrete bridges – Design and detailing rules	UK NA to BS EN 1992-2:2005
<u>UK National Annex to Eurocode 3: Design of Steel Structures –</u>	
Part 1-1: General rules and rules for buildings	UK NA to BS EN 1993-1-1:2005
Part 1-5: Plated structural elements	UK NA to BS EN 1993-1-5:2006
Part 1-8: Design of joints	UK NA to BS EN 1993-1-8:2005
Part 1-9: Fatigue	UK NA to BS EN 1993-1-9:2005
Part 1-10: Material toughness and through-thickness properties	UK NA to BS EN 1993-1-10:2005
Part 1-11: Design of structures with tension components	UK NA to BS EN 1993-1-11:2006
Part 2: Steel bridges	UK NA to BS EN 1993-2:2006
<u>UK National Annex to Eurocode 4: Design of Composite Steel and Concrete Structures –</u>	
Part 1-1: General rules and rules for buildings	UK NA to BS EN 1994-1-1:2004
Part 2: General rules and rules for bridges	UK NA to BS EN 1994-2:2005
<u>UK National Annex to Eurocode 8: Design of Structures for Earthquake Resistance –</u>	
Part 1: General rules, seismic actions and rules for buildings	UK NA to BS EN 1998-1:2004
Part 2: Bridges	UK NA to BS EN 1998-2:2005

A.4 PUBLISHED DOCUMENTS

Background information to the National Annex to BS EN 1991-1-4 and additional guidance	PD 6688-1-4:2009
Recommendations for the design of structure to BS EN 1991-1-7	PD 6688-1-7:2009
Background paper to the National Annex to BS EN 1992-1 and BS EN 1992-3	PD 6687-1:2010

Recommendations for the design of structures to BS EN 1992-2:2005	PD 6687-2:2008
Background to the National Annex to BS EN 1991-2 Traffic loads on bridges	PD 6688-2:2011
Recommendations for the design of structures to BS EN 1993-1-9	PD 6695-1-9:2008
Recommendations for the design of structures to BS EN 1993-1-10	PD 6695-1-10:2009
Recommendations for the design of bridges to BS EN 1993	PD 6695-2:2008 *Incorporating Corrigendum No. 1*
Background paper to BS EN 1994-2 and the UK National Annex to BS EN 1994-2	PD 6696-2:2007
Recommendations for the design of structures for earthquake resistance to BS EN 1998	PD 6698:2009

APPENDIX B

EXAMPLES OF COMBINATIONS OF ACTIONS

FOR TYPICAL ROAD BRIDGES

CONTENTS

B.1 INTRODUCTION

The combinations of actions that need to be considered for a typical road bridges with linear structural analysis for settlement are given in this appendix with the appropriate γ and ψ factors. Not all effects have been included and in some circumstances additional combinations of actions will need to be considered to allow for other effects such as specific construction loads.

The combinations of actions that typically need to be considered are listed below.

ULS Combinations:

Persistent

Permanent + Prestress + Traffic Leading + Wind*
Permanent + Prestress + Traffic Leading + Thermal
Permanent + Prestress + Wind Leading + Traffic
Permanent + Prestress + Thermal Leading + Traffic
Permanent + Prestress + Bearing Friction

Accidental – During

Permanent + Prestress + Accidental + Traffic Leading +Thermal
Permanent + Prestress + Accidental + Thermal Leading + Traffic

Accidental – After

Permanent + Prestress + Traffic Leading + Wind*
Permanent + Prestress + Traffic Leading + Thermal
Permanent + Prestress + Wind Leading + Traffic
Permanent + Prestress + Thermal Leading + Traffic

Seismic

Permanent + Prestress + Seismic+ Traffic

SLS Combinations:

Characteristic

Permanent + Prestress + Traffic Leading + Wind*
Permanent + Prestress + Traffic Leading + Thermal
Permanent + Prestress + Wind Leading + Traffic
Permanent + Prestress + Thermal Leading + Traffic
Permanent + Prestress + Bearing Friction

Frequent

Permanent + Prestress + Traffic Leading + Wind*
Permanent + Prestress + Traffic Leading + Thermal
Permanent + Prestress + Wind Leading + Traffic
Permanent + Prestress + Thermal Leading + Traffic

Quasi-Permanent

Permanent + Prestress + Thermal

Additional Combinations for Hong Kong Only:

Crack Width Verification Combination

Permanent + Prestress + Traffic

Tensile Stress Verification Combination for Prestressed Concrete Members – Case 1: No Tensile Stress Permitted

Permanent + Prestress + Traffic

Tensile Stress Verification Combination for Prestressed Concrete Members – Case 2: Tensile Stress Permitted

Permanent + Prestress + Wind Leading + Traffic
Permanent + Prestress + Thermal Leading + Traffic
Permanent + Prestress + Traffic Centrifugal
Permanent + Prestress + Bearing Friction

* Where road traffic is considered to be simultaneous with wind, the combination value $\psi_0 F_{wk}$ of the wind action on the bridge and on the vehicles should be limited to a value F_w^* as specified in Clauses 3.4.2.2(1) of this Manual.

B.2 ULS COMBINATIONS OF ACTIONS

Table B.1 – Examples of ULS Combinations of Actions for Typical Road Bridges

			ULS Persistent																
					Traffic Leading + Wind[6]	Traffic Leading + Thermal	Wind Leading	Thermal Leading	Bearing Friction	Traffic Leading + Wind[6]		Traffic Leading + Thermal		Wind Leading		Thermal Leading		Bearing Friction	
			γ_{Sup}	γ_{Inf}	ψ	ψ	ψ	ψ	ψ	$\gamma_{Sup}\psi$	$\gamma_{Inf}\psi$	$\gamma_{Sup}\psi$	$\gamma_{Inf}\psi$	$\gamma_{Sup}\psi$	$\gamma_{Inf}\psi$	$\gamma_{Sup}\psi$	$\gamma_{Inf}\psi$	$\gamma_{Sup}\psi$	$\gamma_{Inf}\psi$
Self Weight		Concrete	1.35	0.95	1	1	1	1	1	1.35	0.95	1.35	0.95	1.35	0.95	1.35	0.95	1.35	0.95
		Steel	1.20	0.95	1	1	1	1	1	1.20	0.95	1.20	0.95	1.20	0.95	1.20	0.95	1.20	0.95
SDL		Surfacing[2]	1.20	0.95	1	1	1	1	1	1.20	0.95	1.20	0.95	1.20	0.95	1.20	0.95	1.20	0.95
		Other SDL	1.50	0.95	1	1	1	1	1	1.50	0.95	1.50	0.95	1.50	0.95	1.50	0.95	1.50	0.95
Other Permanent		Settlement	1.20	0	1	1	1	1	1	1.20	0	1.20	0	1.20	0	1.20	0	1.20	0
		Shrinkage	1.00	1.00	1	1	1	1	1	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
		Prestress	1.10	0.90	1	1	1	1	1	1.10	0.90	1.10	0.90	1.10	0.90	1.10	0.90	1.10	0.90
Variable Actions	**Traffic Actions[3, 4 & 5]**	gr1a TS	1.35	0	1	1	0.75	0.75	0	1.35	0	1.35	0	1.01	0	1.01	0		
		gr1a UDL	1.35	0	1	1	0.75	0.75	0	1.35	0	1.35	0	1.01	0	1.01	0		
		gr1a Footway	1.35	0	1	1	0.40	0.40	0	1.35	0	1.35	0	0.54	0	0.54	0		
		gr1b	1.35	0	1	1	0	0	0	1.35	0	1.35	0	0	0	0	0		
		gr2	1.35	0	1	1	0	0	0	1.35	0	1.35	0	0	0	0	0		
		gr3	1.35	0	1	1	0	0	0	1.35	0	1.35	0	0	0	0	0		
		gr4	1.35	0	1	1	0	0	0	1.35	0	1.35	0	0	0	0	0		
		gr5	1.35	0	1	1	0	0	0	1.35	0	1.35	0	0	0	0	0		
		gr6	1.35	0	1	1	0	0	0	1.35	0	1.35	0	0	0	0	0		
	Other Variable Actions[4]	Wind[5] (Simplified or Full Procedure)	1.55 or 2.10	0	0.50		1			0.78 or 1.05	0			1.55 or 2.10	0				
		Thermal	1.45	0		0.60		1				0.87	0			1.45	0		
		Accidental	1.00	0															
		Seismic	1.00	0															
		Bearing Friction	1.30	0					1									1.30	0

1. Permanent actions and leading variable actions (or accidental /seismic actions) do not need to be reduced by combination factors ψ in the combination of actions. For convenience of presentation, they are indicated to have a combination factor $\psi = 1$ in the table.
2. The nominal value of surfacing is varied in accordance with the UK NA to BS EN 1991-1-1 Table NA.1. This requires +55% and -40% of the nominal value to be considered for any combination.
3. Either gr1a or gr1b or gr2 or gr3 or gr4 or gr5 or gr6 is applied in any one combination.
4. In accordance with BS EN 1990 Clause A2.2.2(2), LM2 and the concentrated load Q_{fwk} on footways need not be combined with any other variable non traffic action.
5. In accordance with BS EN 1990 Clause A2.2.2(3), gr2, gr3 and gr4 need not be combined with wind actions.
6. Where road traffic is considered to be simultaneous with wind, the combination value $\psi_0 F_{wk}$ of the wind action on the bridge and on the vehicles should be limited to a value F_w^* as specified in Clauses 3.4.2.2(1) and 3.4.3.3(1) of this Manual.
7. The γ factors are those for the STR/GEO Set B.

The ULS transient combinations are identical to the ULS persistent combination unless modifications are made for a specific project.

B.3 ACCIDENTAL AND SEISMIC COMBINATIONS OF ACTIONS

Table B.2 – Examples of Accidental and Seismic Combinations of Actions for Typical Road Bridges

			Accidental						**Seismic**
			During		**After**				
			Traffic Leading	Thermal Leading	Traffic Leading + Wind[6]	Traffic Leading + Thermal	Wind Leading	Thermal Leading	
			ψ	ψ	ψ	ψ	ψ	ψ	ψ
Self Weight		Concrete	1	1	1	1	1	1	1
		Steel	1	1	1	1	1	1	1
SDL		Surfacing[2]	1	1	1	1	1	1	1
		Other SDL	1	1	1	1	1	1	1
Other Permanent		Settlement	1	1	1	1	1	1	1[7]
		Shrinkage	1	1	1	1	1	1	1[7]
		Prestress	1	1	1	1	1	1	1
Variable Actions	**Traffic Actions[3, 4 & 5]**	gr1a TS	0.75	0	0.75	0.75	0	0	0.20[8]
		gr1a UDL	0.75	0	0.75	0.75	0	0	0.20[8]
		gr1a Footway	0.40	0	0.40	0.40	0	0	0
		gr1b	0.75	0	0.75	0.75	0	0	0
		gr2	0	0	0	0	0	0	0
		gr3	0.40	0	0.40	0.40	0	0	0
		gr4	0	0	0	0	0	0	0
		gr5	0	0	0	0	0	0	0
		gr6	0	0	0	0	0	0	0
	Other Variable Actions[4]	Wind[5]			0		0.20		
		Thermal	0.50	0.60		0.50		0.60	0.50[7]
		Accidental	1	1					
		Seismic							1

1. Permanent actions and leading variable actions (or accidental /seismic actions) do not need to be reduced by combination factors ψ in the combination of actions. For convenience of presentation, they are indicated to have a combination factor ψ = 1 in the table.
2. The nominal value of surfacing is varied in accordance with the UK NA to BS EN 1991-1-1 Table NA.1. This requires +55% and -40% of the nominal value to be considered for any combination.
3. Either gr1a or gr1b or gr2 or gr3 or gr4 or gr5 or gr6 is applied in any one combination.
4. In accordance with BS EN 1990 Clause A2.2.2(2), LM2 and the concentrated load Q_{fwk} on footways need not be combined with any other variable non traffic action.
5. In accordance with BS EN 1990 Clause A2.2.2(3), gr2, gr3 and gr4 need not be combined with wind actions.
6. Where road traffic is considered to be simultaneous with wind, the combination value $\psi_0 F_{wk}$ of the wind action on the bridge and on the vehicles should be limited to a value F_w^* as specified in Clauses 3.4.2.2(1) and 3.4.3.3(1) of this Manual.
7. Action effects due to imposed deformations only need to be included in the seismic combination in the case of bridges in which the seismic action is resisted by elastomeric laminated bearings.
8. ψ factors for quasi-permanent values of mass corresponding to traffic actions are given in Clause 4.9(1) of this Manual.
9. No γ factors need to be applied for accidental or seismic combinations.

B.4 SLS COMBINATIONS OF ACTIONS

Table B.3 – Examples of SLS Combinations of Actions for Typical Road Bridges

			Characteristic					**Frequent**						**Tensile Stress Verification for Prestressed Concrete Members**				
															Case 2: Tensile Stress Permitted			
			Traffic Leading + Wind[9]	Traffic Leading + Thermal	Wind Leading	Thermal Leading	Bearing Friction	Traffic Leading + Wind[9]	Traffic Leading + Thermal	Wind Leading	Thermal Leading	**Quasi - permanent – Thermal**	**Crack Width Verification[2]**	**Case 1: No Tensile Stress Permitted**	Wind Leading	Thermal Leading	Centrifugal	Bearing Friction
			ψ	ψ	ψ	ψ	ψ	ψ	ψ	ψ	ψ	ψ	ψ	ψ	ψ	ψ	ψ	ψ
Self Weight		Concrete	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
		Steel	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
SDL		Surfacing[3]	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
		Other SDL	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
Other Permanent		Settlement	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
		Shrinkage	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
		Prestress	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
Variable Actions	**Traffic Actions[4, 5 & 6]**	gr1a TS	1	1	0.75	0.75		0.75	0.75	0	0	0	1	1	0.75	0.75		
		gr1a UDL	1	1	0.75	0.75		0.75	0.75	0	0	0	1	1	0.75	0.75		
		gr1a Footway	1	1	0.40	0.40		0.40	0.40	0	0	0	1	1	0.40	0.40		
		gr1b	1	1	0	0		0.75	0.75	0	0	0	1[7]	1[7]	0	0		
		gr2	1	1	0	0		0	0	0	0	0	0	0	0	0	1	
		gr3	1	1	0	0		0.40	0.40	0	0	0	0	0	0	0		
		gr4	1	1	0	0		0	0	0	0	0	0	0	0	0		
		gr5	1	1	0	0		0	0	0	0	0	1[8]	1[8]	1	1		
		gr6	1	1	0	0		0	0	0	0	0	0	0	0	0	1	
	Other Variable Actions[5]	Wind[6]	0.50		1			0		0.20					1			
		Thermal		0.60		1			0.50		0.60	0.50				1		
		Accidental																
		Seismic																
		Bearing Friction					1											1

1. Permanent actions and leading variable actions (or accidental /seismic actions) do not need to be reduced by combination factors ψ in the combination of actions. For convenience of presentation, they are indicated to have a combination factor ψ = 1 in the table.
2. Traffic Actions for crack width verification combination do not need to be reduced by combination factors ψ. For convenience of presentation, they are indicated to have a combination factor ψ = 1 in the table.
3. The nominal value of surfacing is varied in accordance with the UK NA to BS EN 1991-1-1 Table NA.1. This requires +55% and -40% of the nominal value to be considered for any combination.
4. Either gr1a or gr1b or gr2 or gr3 or gr4 or gr5 or gr6 is applied in any one combination.
5. In accordance with BS EN 1990 Clause A2.2.2(2), LM2 and the concentrated load Q_{fwk} on footways need not be combined with any other variable non traffic action.
6. In accordance with BS EN 1990 Clause A2.2.2(3), gr2, gr3 and gr4 need not be combined with wind actions.
7. For cantilever slabs and the top flanges in beam-and-slab, voided slab and box-beam construction, gr1b should be considered.
8. The axle loads of the SV196 vehicle (See also Clause 3.7.2.4 of this Manual) should be multiplied by 0.56.
9. Where road traffic is considered to be simultaneous with wind, the combination value $\psi_0 F_{wk}$ of the wind action on the bridge and on the vehicles should be limited to a value F_w^* as specified in Clauses 3.4.2.2(1) and 3.4.3.3(1) of this Manual.
10. No γ factors need to be applied for SLS combinations. For the Quasi – Permanent combinations no leading effects need to be specified.

APPENDIX C

BACKGROUND TO THE WIND ACTION PROVISIONS

FOR HIGHWAY STRUCTURES AND RAILWAY BRIDGES

CONTENTS

C.1 INTRODUCTION

This appendix has been prepared to provide the designer with background information on the derivation of the wind loading clauses in this Manual.

This Manual has been written for Hong Kong conditions but embodies many of the basic principles of BS EN 1991-1-4. This Appendix highlights the reasons for the differences in the wind loading between this Manual and BS EN 1991-1-4, the UK NA to BS EN 1991-1-4 and PD6688-1-4.

C.2 THE HONG KONG WIND CLIMATE

The Hong Kong wind climate is characterized by its position in one of the most severe typhoon regions in the world. The typhoon season affects Hong Kong for roughly half the year, with the wind climate being dominated by the winter monsoons in most of the remaining portion. The climate can be considered as bi-modal. As far as the design of bridges are concerned, the typhoon environment produces the most onerous loadings, however the monsoon season should not be neglected, particularly when considering response due to vortex induced excitation.

The wind structure of a typhoon is significantly different from that of the wide-front, low pressure (depression) storms of Western Europe and the Western seaboard of the United States of America. The wind speeds are significantly higher in a typhoon as are the intensities of turbulence. Gust factors will also tend to be higher. Hong Kong is also characterized by its very significant topography and by the substantial shielding that can exist in urban environments. Both serve to increase the turbulence of the wind and may require greater attention to local wind effects at ground level, particularly those affecting pedestrian comfort.

C.3 WIND LOADING PROVISIONS

Wind loading provisions in this Manual have been written to cover the design of conventional pedestrian, highway and railway bridges in Hong Kong, constructed from steel, reinforced or prestressed concrete or from steel/reinforced concrete composite sections.

The designer of bridges that are inherently more flexible, such as cable-stayed or suspension bridges, can use the criteria contained in Clause 3.4.3 of this Manual but due care must be taken in correctly assessing the dynamic effects of such light and flexible structures. Particular attention is drawn to footbridges of this type, which may need to be designed to criteria more onerous than those contained in this Manual.

C.4 BACKGROUND TO CLAUSE 3.4

This Manual provides procedures for the determination of quasi-static wind loading on highway and railway structures. The procedures are split into two distinct approaches based on span length and the economic importance of the route.

The basis behind this split is twofold: firstly to provide a simple method of analysis for simpler structures; and secondly to recognize the importance of the costs and consequences of failure.

The simpler method in Clause 3.4.2 results in the provision of an invariant peak velocity pressure and a lower partial load factor. This approach is changed for bridges conforming to the requirements of Clause 3.4.3 where the concept of loaded length is introduced, i.e. the intensity of the loading is a function of the length of loaded part. The partial load factor is also increased, to reflect the higher consequences of failure of these bridges.

The speed of the gust is related to its duration. The higher the speed of the gust, the shorter its duration and the less of the structure it envelops. Thus the effective speed of a gust that encompasses the entire span of a bridge will be less than that which covers only a part of the span.

Clause 3.4.3 provides a table of dynamic wind pressures (Table 3.9) in which wind pressures of varying intensity are given for a range of loaded lengths and structure heights. The effect of the size of the gust can be seen in the decreasing magnitude of the wind pressure with increasing length.

C.4.1 Velocity Pressure for Wind Leading Combinations

The procedures given in Clause 3.4.3 provide the designer with the means of assessing the influence of gust loads on the design of the structure. For each bridge element the designer is required to determine the loaded length, which maximizes the wind load on the element. For simpler line-like structures (e.g. simply supported viaducts), the appropriate loaded lengths will be self evident. For other structures, an influence line analysis may be required.

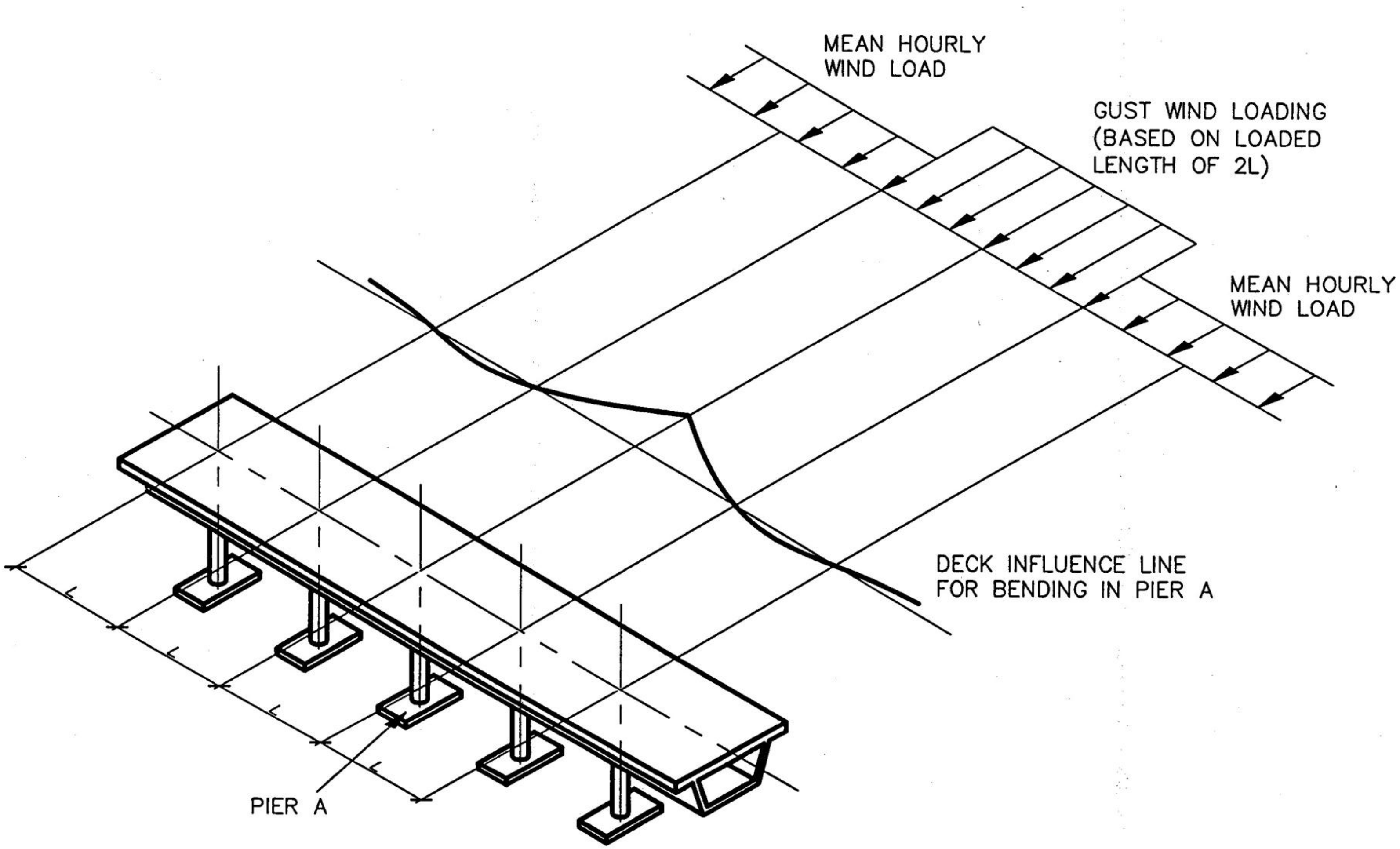
The full gust load is applied to the positive part of the influence line, which causes the greatest load effect. For the remainder of the structure, the mean hourly wind load component should be applied, as described in Clause 3.4.3.2. The procedure is illustrated in Figure C1. This procedure needs to be repeated for each appropriate element of the bridge.

C.4.2 Velocity Pressure for Traffic Leading Combinations

The influence of wind forces on bridges loaded with traffic also needs to be considered. Although the peak velocity pressure is considerably reduced with the lower wind speeds that are co-incident with traffic loads, the additional projected area of high-sided vehicles may result in a significant increase in overall wind load. The combination of the wind and traffic loads may govern the design of some elements of the bridge.

Clause 3.4.3.3 provides the means for assessing the wind load coincident with traffic load. Generally the wind load is unlikely to exceed greatly the equivalent mean hourly wind loading. Wind speeds in excess of the mean hourly wind speed of 35m/s make driving conditions very difficult and it is unlikely that significant volumes of traffic will be present on most bridge structures at higher wind speeds.

Figure C1 – Use of Influence Lines to Calculate Gust Wind Loads



C.4.3 Overturning Effects

For structures where wind loads may result in overturning effects in piers or foundation, consideration should be given to the combinations of wind load with traffic. The increased projected area from high-sided vehicles may be sufficient to increase the overall overturning effect on the structure. The most adverse distribution of traffic should be considered and the reduced traffic action given in Clause 3.4.3.5 should be used if the vertical load relieves the overturning effect. The reduced traffic action represents essentially unloaded vehicles. A reduced γ_Q is also applicable.

C.4.4 Dynamic Response Procedure

The provisions for the calculation of aerodynamic effects on bridges are taken from clause NA.2.49 of the UK NA to BS EN 1991-1-4 and PD6688-1-4 Annex A. The designer is directed to undertake an three assessments of the structure :

(1) Review the along wind response (given in clause NA.2.49.1) – highway and railway ridges of less than 200m span do not normally require explicit allowance for dynamic response in the along wind direction.

(2) Review vertical wind response (given in clause NA.2.49.2) – it is same as the turbulence response originally defined in BD49/01. Dynamic effect of turbulence response can be ignored provided either: (a) The fundamental frequencies in both bending and torsion > 1.0Hz; or (b) $P(z) \leq 1.0$. If condition (a) and (b) are not satisfied, the dynamic effects of turbulence response should be considered and specialist advice sought.

(3) Review aerodynamic stability (given in clause NA.2.49.3) - the aerodynamic susceptibility parameter, P_b is determined based on span length, deck weight and width and material type (to assess damping). This parameter is intended to test the sensitivity of the bridge deck to potential problems such as vortex shedding, strong-excitation aerodynamic instabilities including galloping and flutter. For those structures found to be relatively insensitive, no further assessment will be required. For those bridges with moderate sensitivity, the provision of PD6688-1-4 may be applied. For bridges that are shown to be highly sensitive (P_b>1.0) and beyond the scope of PD6688-1-4, wind tunnel testing will be required.

Generally the types of bridges requiring wind tunnel testing will be limited to longer span bridges with spans in excess of 200m, bridges with lightweight decks (steel orthotropic decks) or those with unusual structural support systems.

The background to the physical processes considered in the aerodynamic response of bridges is given in Appendix D.

Revisions have been made to the rules given in PD6688-1-4 for assessing aerodynamic effects. The hourly mean wind velocity for relieving area $v_r(z)$, the peak wind velocity $v_d(z)$, the hourly mean velocity pressure $q'(z)$ and the peak velocity pressure $q_p(z)$ are directly specified in this Manual based on hourly mean wind speed to suit the different gust model appropriate to the wind climate of Hong Kong. Therefore, the provisions in PD6688-1-4 based on 10-minute mean wind speeds have been reviewed and modified equations have been specified in this Manual based on hourly mean wind speed and wind factors appropriate to determine the aerodynamic effect for highway structure design in Hong Kong.

C.4.5 Force Coefficients

The designer is required to calculate the wind loads on the bridge based upon force coefficient obtained from data presented in BS EN 1991-1-4 and the UK NA to the BS EN 1991-1-4. Force coefficients have been tabulated for most conventional deck cross sections. For those sections not covered by BS EN 1991-1-4 and the UK NA to the BS EN 1991-1-4, the designer must either make conservative assumptions based upon the provisions within BS EN 1991-1-4 and the UK NA to the BS EN 1991-1-4 or seek data from wind tunnel tests. It is admissible to use data which have been obtained from tests on other bridges. If wind tunnel tests are proposed, the recommendations given in PD6688-1-4 and Appendix E of this Manual should be applied.

For lightweight longer span structures, the vertical component of wind load may be sufficient to affect the design of the deck. Particular attention is drawn to the effects of cross wind turbulent response for such structures which are likely to be more sensitive to turbulent response. The method for assessing if a bridge is sensitive to turbulent response is given in Clause 3.4.4 where the designer is directed to the provisions given in the UK NA to BS EN 1991-1-4 and PD6688-1-4.

C.4.6 ULS Partial Factors

The increased load factor reflects to an extent the increasing level of structural reliability sought from longer span structures. The factor does not in any way represent an enhanced loading factor to deal with structural response, but provides a measure of the increased dispersion of extreme wind speeds in a typhoon.

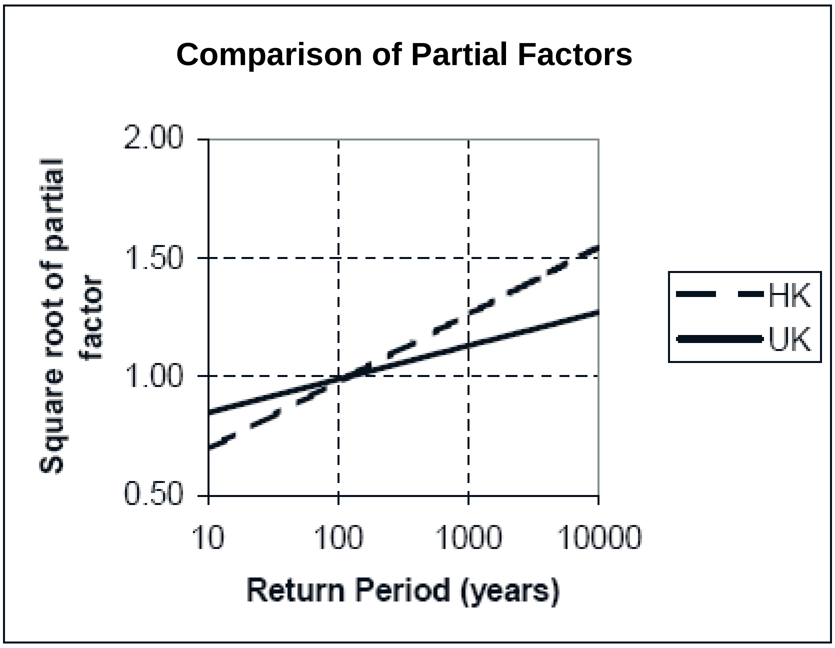
Current design procedures use partial safety factors to provide an acceptable probability of failure - that is a required safety level. In effect the partial safety factor on windloading decreases the probability of exceedence of the notional wind speed used in design from the normal design value of a 120 year return period (or probability of exceedence in anyone year of 0.0083) to a more appropriate level. For structures designed to Clause 3.4.3, this value exceeds 2400 years (or a probability of exceedence in any one year of 4.2 x 10^{-4}).

For the United Kingdom the factor to achieve this is about 1.2 on wind speed (1.4 on load) but in Hong Kong, due to the higher dispersion of typhoon winds, the factor needed becomes about 1.37 on wind speed (1.9 on load). This is shown diagrammatically in Figure C2. The equations for the two distributions are :

$\gamma_{fL} = [1 + 0.124 \ 1n \ (T/120)]^2$ for Hong Kong
$\gamma_{fL} = [1 + 0.068 \ 1n \ (T/120)]^2$ for the UK

where T = Return period in years

Figure C2 – Partial load factors on wind



It should be noted that these predictions are based on extreme value models based on Fisher-Tippette Type 1 or Gumbell distributions on maximum wind speed. These provide values that are more conservative than models based on distributions on the square of the maximum wind speed, as used in temperate climatic zones.

The factor has been reduced for structures with, inter alia, spans of less than 100 m to 1.4. This factor, when applied to the constant peak velocity pressure of 3.8kN/m^2 in exposed terrain, produces the same overall factored load for structures of span length of 100 m designed to Clause 3.4.3. Thus, the same overall level of reliability has been attained and there is no sudden jump in the loads applied to structures below and above 100 m in span length.

The intention of this two-stage approach is to provide greater overall reliability for major structures. This reflects the increased cost of replacement of larger structures, the increased economic and social importance of major fixed links, and the greater loss to the community should a structure like Tsing Ma bridge be forced to close due to wind damage. It should be noted that the two approaches provide approximately the same factored wind load for spans of 100 m.

The introduction of the Eurocodes represents a major change to the previous Hong Kong practice for the design of bridges. For reasons of safety and economy, it is important to ensure that design to the BS EN 1991-1-4, along with the modification adopted in this Manual, result broadly in the same level of overall safety and reliability as is implicit in the partial factors adopted in previous SDMHR. The partial load factor γ_Q has been adjusted to ensure a consistent level of reliability compare to previous SDMHR. This has involved derivation of the γ_Q factors taking into account of the following :

(1) Use of the γ_{fL} factor adopted in previous SDMHR

(2) Inclusion of the additional γ_{f3} factor of 1.1 used for determining design load effects in previous SDMHR from BD37/01.

Care should be taken in designing 'landmark' structures that may be more prone to wind load effects. If the designer is in doubt over the loading requirements for a structure because of a perceived need for a higher structural reliability, the designer should consult the Chief Highways Engineer/Bridges and Structures for advice.

APPENDIX D

BACKGROUND ON AERODYNAMIC EFFECTS ON BRIDGES

CONTENTS

D.1 INTRODUCTION

The majority of bridge structures already constructed in Hong Kong will exhibit no significant aerodynamic response. However, advances in materials and design techniques are resulting in lighter and longer span structures that are increasingly susceptible to aerodynamic response. The following section presents the background to some of these aerodynamic phenomena and the methods by which they should be assessed.

The majority of shorter span structures will not be prone to vibrations caused by effects generated by the wind. Such structures can be considered as being effectively rigid or 'quasi-static'. The load that is imposed on the structure is proportional to the square of the gust wind speed. The structure does not respond to the time-varying nature of the wind and the load generated is treated as a static effect. This is the basis behind the calculation of basic wind load effects within this Manual.

As the bridge spans increase, the likelihood of their dynamic response increases. This is a function of a decrease in the frequency of vibration, down to levels that correspond with the peak incidence of energy in the wind spectrum. Thus, consideration may need to be given to their dynamic response to wind effects, which involves considering additional mechanisms to those that are adequate for quasi-static structures.

There are three principal aerodynamic effects that would need to be considered for such bridges :

(1) divergent response, including galloping and flutter;

(2) vortex excitation; and

(3) turbulence response.

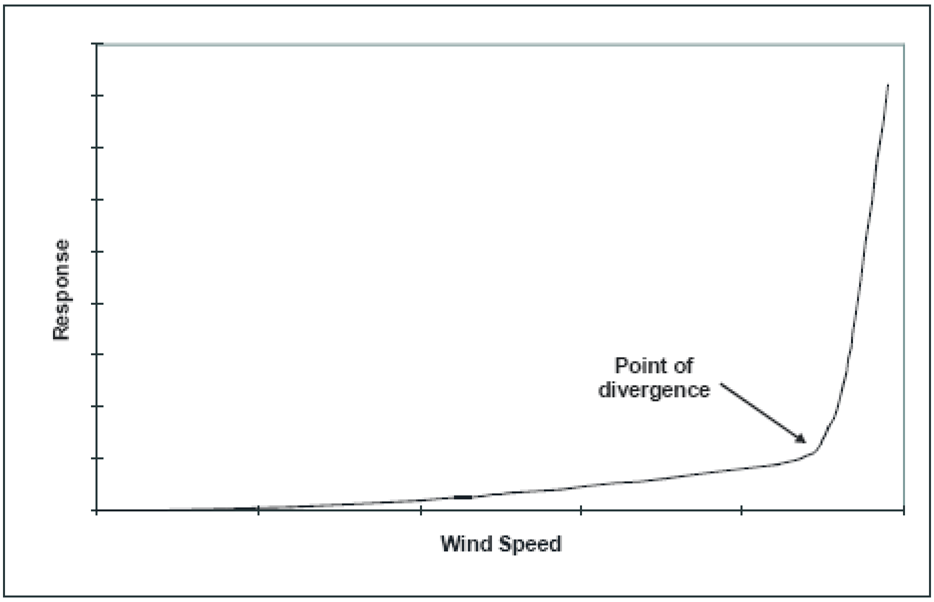
These three mechanisms will be considered separately at first.

D.2 DIVERGENT RESPONSE

Some bridge sections exhibit divergent response characteristics which must be avoided, through re-design, as they can lead to catastrophic failure. A notable example is the Tacoma Narrows bridge collapse in 1940. An example of such a response is shown in Figure D1 where the response increases rapidly with increase of wind speed.

Divergent responses include both galloping and flutter. Galloping is rarely found to be a problem for bridge decks but may be problematic in free-standing bridge towers with combinations of circular and flat forms. Sections that resemble a 'D' shape have been shown to be particularly responsive. The mechanism for galloping can be identified in wind tunnel testing, however its propensity can be assessed using UK NA to BS EN 1991-1-4 Clause 2.49 with PD6688-1-4 Annex A.

Figure D1 – Divergent response



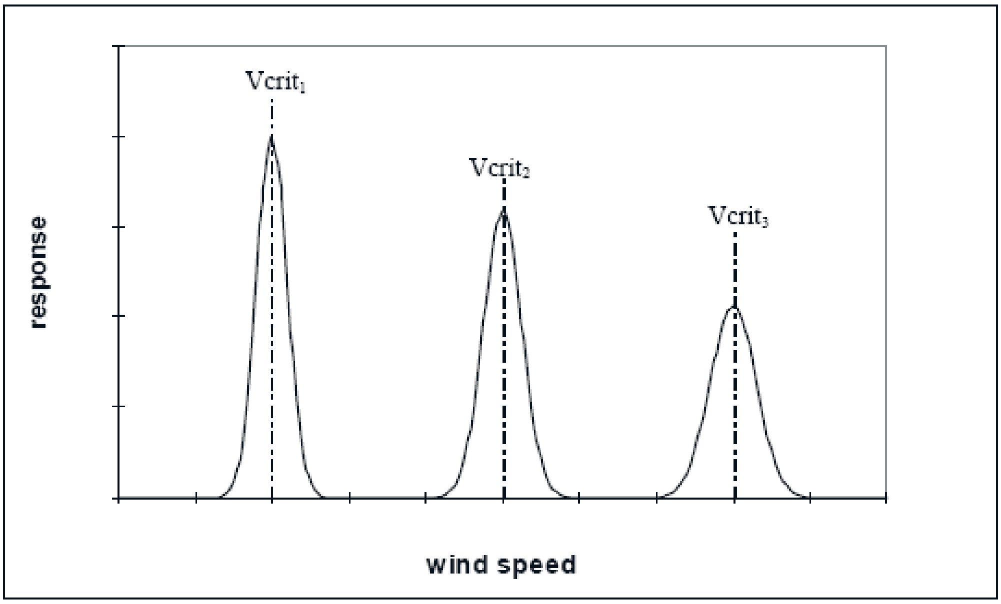
A similar type of response is due to flutter, which may involve either a coupled oscillation, i.e. simultaneous vertical and torsional motion at a given relative amplitude and phase (classical flutter) or a purely torsional response (torsional flutter). It originates from forces caused by the relative motion between the approaching wind and the deck cross section. The motions are very sensitive to the shape of the cross-section, in particular, the leading edge details. Often, edge details need to be revised following wind tunnel tests to provide aerodynamically shaped edge fairings.

In design it must be ensured that the onset of such a response is well above the likely wind speeds to occur within the lifetime of the bridge. This is achieved by establishing a limiting wind speed, which has a very low probability of occurrence. Typically this will be described as a gust wind speed of a duration equal to the length of time taken to build up such divergent amplitudes of vibration, with a low probability of occurrence in a return period of 120 years.

D.3 VORTEX EXCITATION

The periodic shedding of vortices, alternately from the upper and lower surfaces of the deck, causes periodic fluctuations of the aerodynamic forces on the structure. When the frequency of vortex shedding coincides with a natural frequency of the bridge, oscillations build up and resonant conditions occur at a specific ‘critical’ wind speed. A typical response is shown in Figure D2.

Figure D2 – Vortex excitation



It should be noted here that the load effects due to vortex excitation, when combined with the other wind effects at the relevant critical wind speed, need to be considered in design. Clearly if the critical wind speed is below the design wind speed (for the first mode, f1, this is usually the case) then vortex shedding will occur and the partial factor needed to ensure adequate safety has to cater for the uncertainty in predicting the magnitude of response only, i.e. it is not a function of the wind environment. Thus, a γ_Q factor of 1.32 has been adopted in this Manual for this purpose. This is independent of the wind climate.

D.4 TURBULENCE RESPONSE[1]

The design of modern long span bridges presents some particular problems in the calculation the wind forces on the deck, towers and cables. Generally engineers are familiar with methods for coping with the in-line effects of wind, and relatively straight forward rules are provided in codes of practice such as UK NA to BS EN 1991-1-4 and PD6688-1-4.

However, turbulent response requires a more detailed treatment to get a true measure of the effective wind force components. In general, cross wind or vertical response is more significant in longer span bridge structures than in-line or horizontal response, although inline response should not be ignored if lateral frequencies are less than 1 Hz.

The cross-wind forces are generated by changes in the angle of the incident wind from horizontal due to the presence of large gusts. There are a number of key factors that can exacerbate cross-wind response and these include the followings :

(1) Modern long-span bridge decks tend to be much lighter than conventional short-span girder structures, thus the effect of any vertical wind loading becomes more significant.

(2) The trend to design bridges with a high width to depth aspect ratio increases the sensitivity of the bridge to any incidence in the flow.

[1] The term 'Buffeting Response' is also in common usage

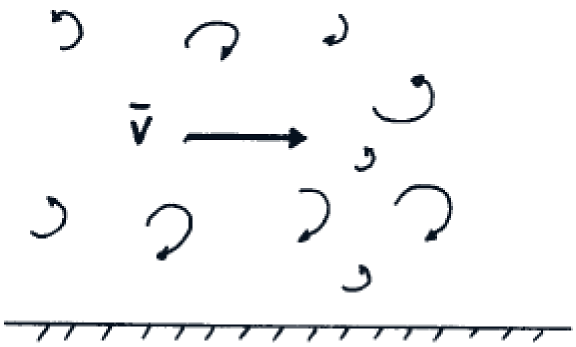
(3) If the frequency of the structure lies within the peak of the wind power spectrum, this vertical response may be amplified significantly giving rise to narrow band response.

It is worth noting that turbulent responses are not strictly aero-elastic because although the wind loads may interact with the stiffness to produce enhanced (resonant) responses, the motion of the bridge does not in turn affect the applied wind loads (this is usually considered to be the defining characteristic of an aero-elastic response). Thus although at first appearance the calculation of cross-wind responses may seem to be fairly complex matter, the underlying methods that need to be applied and understood are exactly the same as those for in-line wind response.

In order to fully understand the nature of structural responses to turbulent wind loading, it is useful to go back to first principals.

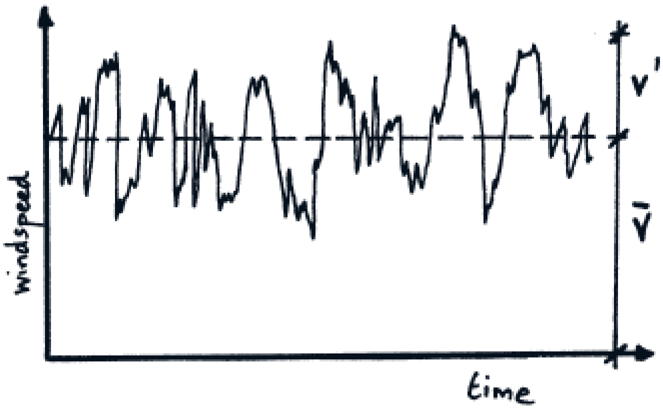
In general the wind does not blow at a uniform speed. Buildings and bridges lie deep within the earth's boundary layer where the wind is turbulent and constantly varying, with fluctuating components in all three directions, i.e. along the wind, vertically and horizontally. In many instances the along wind component dominates but for flexible structures having a relatively large aspect ratio, such as is seen on many modern bridges, this is not always the case. It is convenient to think of the structure of the wind as being represented by large vertical wind packets known sometimes as eddies (blocks of circulating air) that are swept downstream at the velocity of the mean wind. These eddies are caused by friction between the air-stream and the ground (and any other obstacles) and it is this friction that results in the development of the earth's boundary layer hence the familiar wind profile with height used in codes.

Figure D3 – Wind Turbulence



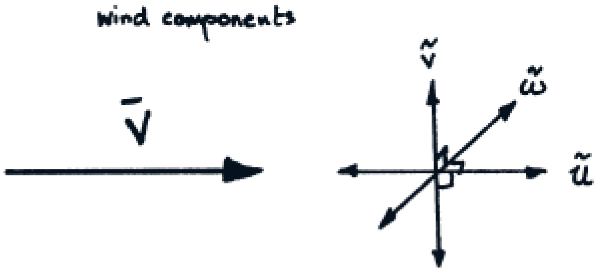
It can be seen from the turbulent nature of the wind shown pictorially in Figure D3 that in the in-line wind direction the observed wind speed is comprised of two components, the mean wind speed V , plus the fluctuating component v' (Figure D4).

Figure D4 – Time History of Wind Speed



However, in general the particles of air circulate in both the horizontal and vertical planes and so there are fluctuating components of wind velocity in all three directions that are swept downstream at the mean velocity (Figure D5).

Figure D5 – Representation of Fluctuating Wind Components



The in-line, cross-wind horizontal and cross-wind vertical fluctuating components of velocity are all of a similar size (but close to the ground the vertical component becomesless). Each of these components in reality represents a continuous spectrum of particle amplitudes and circulation frequencies and that can conveniently be represented by the spectral density function of wind velocities.

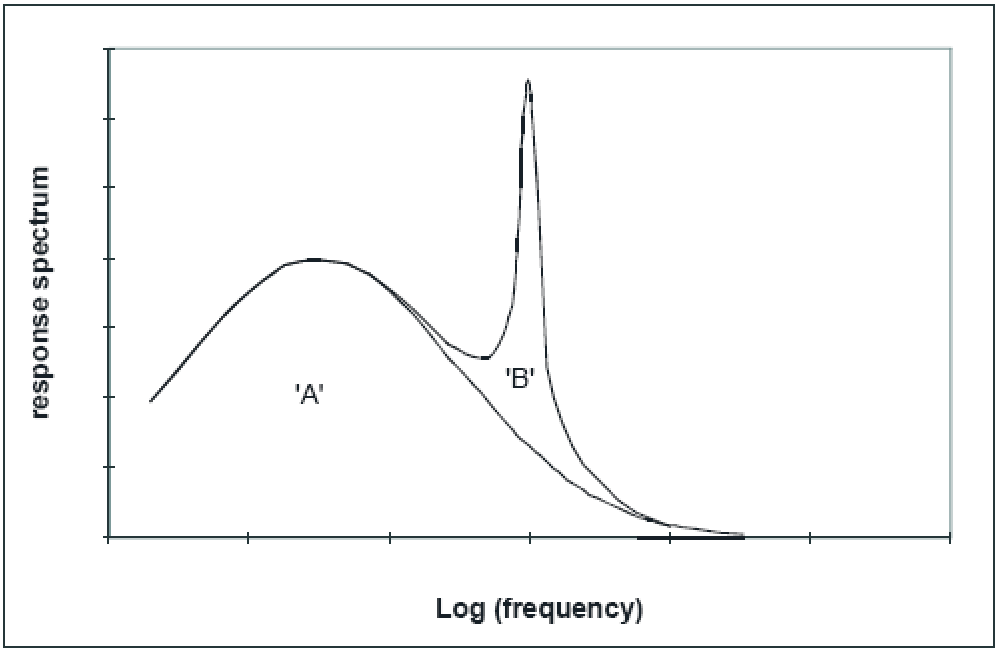
Clearly in the above the maximum in-line component of wind component of wind speed to be expected is always likely to be much larger than the vertical component of velocity, and for many structures the resulting loading is likely to be completely dominated by the in-line term. However in the case of very wide and thin bridge decks the effect of the vertical velocity component can become significant. As an example the Ting Kau bridge deck has an aspect ratio in the region of 25:1, and this is by no means exceptional.

Where the lowest natural frequency of the bridge is well above the range of wind frequencies that have the most energy, the structure behaves in a quasi-static manner to the applied fluctuating loads. That is, the applied wind loads are changing slowly enough so that structure can respond as though the load, which is being applied at each instant in time, were not changing at all; in other words the loading is considered to be quasi-static. For these circumstances, using a few simplifying assumptions, codes of practice can provide means to calculate a system of simple equivalent static patch loads that are able to generate the same net load effects that in reality come from the peak fluctuations of a time varying loading. Clause 2.3.2 of this Manual provides the means to calculate these quasi-static in-line wind loads.

In some circumstances such an approach is not adequate. In particular if the natural frequencies of significant bridge modes occur at frequencies where the wind is energetic then there will be an enhanced structural response due to resonance at and around the mode frequencies in question. It should be noted that enhanced response due to resonance is not an effect that is specific only to cross-wind effects. Such resonant magnification occurs whenever the frequency of the applied loading and modes of the structure match irrespective of the direction of the applied load.

In the absence of any resonant effect the spectrum of the resulting structural responses is represented by the area A in Figure D6 below. In this the area under the response spectrum represents the variance of the fluctuating response term (though has already been pointed out, for such simple quasi-static situations equivalent static patch load methods are able to provide equivalent design load effects more directly).

Figure D6 - Response Spectrum of Structure



When the structural frequency lies within significant regions of the loading spectrum the net response in the mode in question looks more like upper line shown above. In this situation,

(1) for that part of the loading having frequencies below the frequency of the mode the response is very similar to the quasi-static response of A;

(2) for loading components near to the natural frequency the responses are significantly enhanced due to the effects of resonance;

(3) for loading components above the natural frequency responses are small.

It is clear that the area under the response spectrum is larger because of the resonant effect. In order to simplify the above analysis it is often found convenient (though it is not necessary) to divide the area under the net response curve into two regions :

(1) Area A : That part of the response which occurs over a broad-band of frequencies obtained by assuming that the bridge frequency is high, otherwise known as the broadband response. (That is the quasi-static response load effect which is equivalent to the static loading provided by the basic methods given in the code.)

(2) Area B : That part of the response which occurs over a narrow range of frequencies where the responses are enhanced as a result of resonance, often denoted the narrow-band response.

Flexible, and in particular dynamically sensitive, structures will suffer enhanced response compared with a quasi-static structure due to the structural frequency of the bridge matching the peak energy in the wind spectrum.

Clearly the response of the bridge will be magnified from that of a rigid structure and there are standard procedures for accounting for this published in the literature. It should be stressed that the vast majority of bridge structures will not be subjected to any significant narrow band response and accordingly, there are no specific codified procedures for estimating narrow band effects. UK NA to BS EN 1991-1-4 Clause 2.49 with PD6688-1-4 Annex A provides guidance to the designer to determine if a bridge deck is likely to be susceptible to cross wind response. For bridge decks with a vertical bending frequency of less than 1 Hz, the designer is directed to an empirical assessment of the sensitivity of the bridge. If the assessment shows that deck is likely to be sensitive, more detailed analysis is required. The designer should first refer to the Engineering Sciences Data Unit (ESDU) papers, in particular ESDU No. 89049 “Response of Structures to Atmospheric Turbulence: Response to Across-wind Turbulence” for further guidance, however, it should be noted that the calculation of such effects is a specialist task. In such cases it would be conventional to undertake theoretical calculations and calibrate them against wind tunnel test results.

D.5 COMBINATION OF TURBULENT RESPONSES WITH OTHER AERODYNAMIC LOAD EFFECTS

The following terms are used in this Appendix to aid in the description of the method of combining turbulent gust response (buffeting) with other aerodynamic and static effects. The terms are not given in this Manual and are introduced here to be assist with the description of the narrow band and broad band effects :

The net load effect (response) that results in a structure is defined as the simple linear sum of (a) the mean, static response, Fmean, (b) fluctuating load effects due to turbulence, $F_{turbulence}$, and (c) any other self-excited aero-elastic responses, $F_{self\text{-}excited}$, where :

F_{mean} = The load effect that results from the application of static forces corresponding to the steady mean wind speed on a structure. The mean effect is equivalent to the time-averaged value of the response component being considered, i.e. it excludes all fluctuating effects;

$F_{turbulence}$ = The fluctuating load effect (i.e. response) that is generated in a structure due to the time-varying component of the wind (buffeting).

$F_{self\text{-}excited}$ = The fluctuating load effect (i.e. response) that is generated due to self-excited aero-elastic mechanisms such as vortex shedding, flutter or galloping. See also Figure D7.

The net fluctuating load effect due to turbulence, Fturbulence , is in turn defined as the combined effect of the broad band and narrow band dynamic responses, Fb and Fn , where:

F_b = The broad band component of the dynamic response that occurs from a quasi-static application of the fluctuating (buffeting) wind loads, ignoring any resonant response (equivalent to area A in Figure D6 above) ;

F_n = The net additional resonant contribution to the load effect that occurs about a narrow range of frequencies around the frequencies of the modes of interest (area B in Figure D6 above).

And the net additional resonant contribution to the load effect is obtained from the combined effect of the responses in each mode of interest, where :

$F_{n(i)}$ = The resonant contribution from each mode i.

It is important to note that all of the 'F' terms defined above are load effects, that is they are the effects produced by the application of loads. They may thus be forces, moments, displacements or even stresses; they are not, and should not be confused with, the applied loads.

If narrow band (resonant) response is found to be significant, the means of incorporating the results into the overall wind calculation effect needs to be considered. In simplistic terms, it is generally accepted that the broad band and narrow band components are uncorrelated; that implies that instead of summing their load effects (i.e. $F_b + F_n$) they are added using the square root of the sum of the squares (SRSS) method (i.e. $\sqrt{F_b^2 + F_n^2}$).

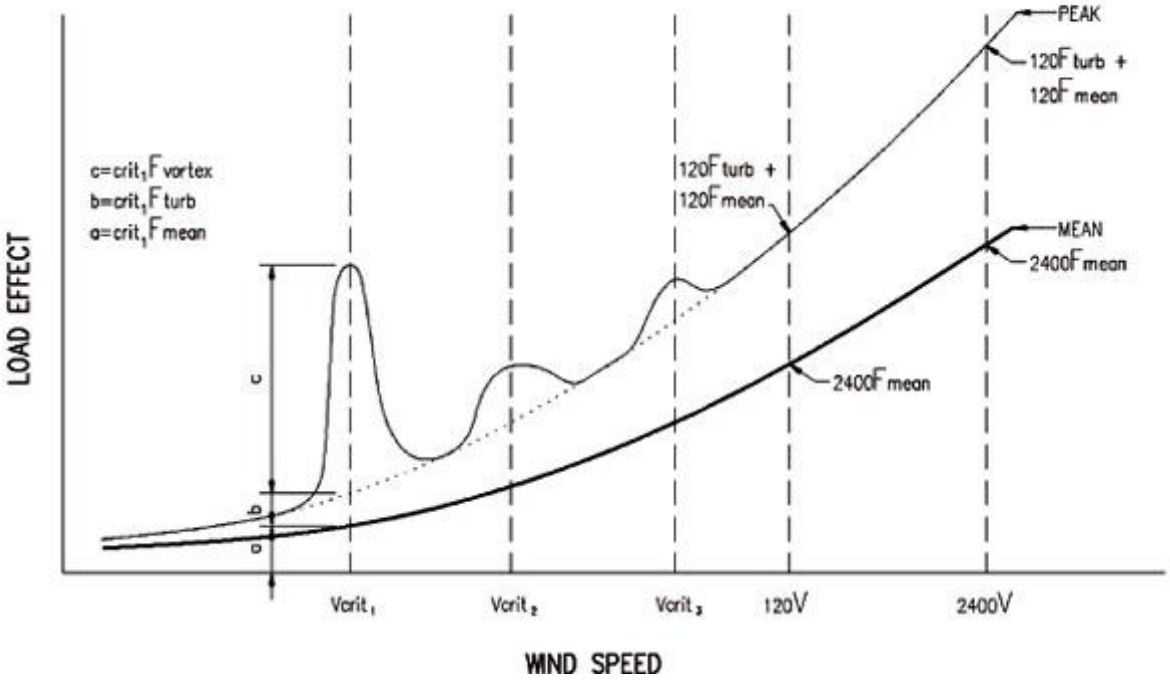
In general for load effects that have a significant contribution from more than one mode the contribution from each mode, i, is also added using the SRSS method though for closely spaced modes other more advanced methods such as the complete quadratic combination (CQC) method may be required.

It is assumed that the possibility of any high wind speed divergent responses such as galloping and flutter will have been eliminated in design (since this Manual requires that their onset is above the limiting wind speed for the bridge). Therefore their effects are not included in this discussion. Furthermore since the high levels of turbulence at high wind speeds acts to eliminate other non-divergent aero-elastic responses (such as vortex shedding) for the most extreme wind speeds it is normally only necessary to consider the combination of the mean wind with the corresponding turbulent gust response.

However at lower wind speeds the combined effect of turbulent response and the remaining self-excited aerodynamic effects (principally vortex shedding) must also be considered. The implications of this on the design procedure follow.

Figure D7 illustrates the typical response of a bridge with increasing wind speed.

Figure D7 – Typical response of a bridge with increasing wind speed



This shows that the response is generally made up of a mean wind component (F_{mean}) and a turbulence component ($F_{turbulence}$) which includes, for a flexible structure, the aerodynamic response from the narrow band as well as the broad band, i.e. $F_{turbulence} = \sqrt{F_b^2 + F_n^2}$).

However at certain critical wind speeds vortex excitation may occur, represented on Figure D7 at critical wind speeds V_{crit1} and V_{crit2} (for the first two modes of vibration). Thus, at the wind speed corresponding to V_{crit1} the response is made up of the mean and turbulence components, crit1Fmean and crit1Fturbulence, as well as the self-excited component, $F_{self\text{-}excited}$, due to vortex shedding, ${}_{crit1}F_{vortex}$.

At 'serviceability' levels the total load effect, at this wind speed, is thus :

$${}_{crit1}F_{mean} + {}_{crit1}F_{turbulence} + {}_{crit1}F_{vortex}$$

While at 'ultimate' levels, using UK NA to BS EN 1991-1-4 Clause 2.49, PD6688-1-4 Annex A and this Manual, the partial factor is γ_Q = 1.32 so the factored load effects, at this wind speed, is thus :

$$1.32\ ({}_{crit1}F_{mean} + {}_{crit1}F_{turbulence} + {}_{crit1}F_{vortex})$$

As has been previously stated, the load factor of 1.2 represents the uncertainty in the calculation of the vortex excitation effects, not in the prediction of the extreme wind speed.

As vortex excitation is generally more severe in low turbulence, frequently crit1Fvortex is obtained from smooth flow conditions but for design purposes is combined conservatively with the mean and turbulence components.

At the higher, extreme wind speeds, critical wind speeds for vortex excitation are unlikely to occur due to the significantly increased turbulence in the wind so that at serviceability one needs to check for the combination of mean and turbulence effects only. The serviceability check is undertaken at the 120 year return period speed ${}_{120}V$ leading to mean and turbulence responses ${}_{120}F_{mean}$ and ${}_{120}F_{turbulence}$ so that the serviceability check is :

$$
\begin{aligned}
& 1.1\ [{}_{120}F_{mean} + {}_{120}F_{turbulence}] \\
= \ & 1.1\ [{}_{120}F_{mean} + \sqrt{\left({}_{120}F_b^2 + {}_{120}F_n^2\right)}\ ]
\end{aligned}
$$

The ultimate limit state requires a partial factor γ_Q in Hong Kong of 2.1 on the 120 year speed.

Thus the ultimate limit state check is given by :

$$
\begin{aligned}
& 2.1\ [{}_{120}F_{mean} + \sqrt{\left({}_{120}F_b^2 + {}_{120}F_n^2\right)}\ ] \\
= \ & 2.1\ {}_{120}F_{mean} + \sqrt{\left[\left(2.1\,{}_{120}F_b\right)^2 + \left(2.1\,{}_{120}F_n\right)^2\right]}\ ]
\end{aligned}
$$

Since it is not possible to describe every possible aero-elastic response mechanism in advance it is important that the rationale behind the combination of wind load effects and the application of partial factors is fully understood. In circumstances not fully considered by this Manual, it is required that engineers should have sufficient expertise to be able to apply techniques that are consistent with the methods embodied in this document.

APPENDIX E

GUIDANCE FOR WIND TUNNEL TESTING

CONTENTS

E.1 INTRODUCTION

Wind tunnel testing forms a vital part of the design and assessment of long span bridges. The aim of the testing is to demonstrate the safety of the bridge in all wind conditions and to determine key parameters for the design of the structure. Wind tunnel testing is expensive and the designer needs to know what tests are required to justify the design and what information should be collected prior to the commencement of testing.

There is also the need to confirm that the tests being carried out are sufficient to maintain the desired safety of the structure for the conditions applicable to the bridge site.

As testing techniques are continually being developed and other procedures, such as Computational Fluid Dynamics, are becoming more reliable, requirements for wind tunnel testing should be kept under constant review. Many useful publications are available which give more extensive details of the theory and practice of wind tunnel testing.

It is important to stress the need for an awareness of the limitations of wind tunnel model tests in general, with special caution in situations where partial or approximate models are used.

E.2 GENERAL REQUIREMENTS FOR WIND TUNNEL TESTING

There are three basic reasons for undertaking wind tunnel tests. Firstly, the tests may be undertaken to obtain static coefficients for use in the basic static design checks for wind or for input to analysis of turbulence response. In the first instance, these data may be obtained from tests undertaken on other structures or, more likely, by application of BD 49/01. However, BD 49/01 does not cover every eventuality and the coefficients provided have a degree of conservatism built in to ensure they apply to a wide range of structures. Thus, if wind forces contribute substantially to the cost of the structure, benefit may be gained in determining the static coefficients more precisely through wind tunnel tests. Clearly, the cost of the wind tunnel tests must be offset in the savings achieved in reducing the structural weight of the bridge.

Secondly, the tests will provide coefficients for checks on vortex excitation effects or divergent amplitude effects. Such tests require dynamic models and can also yield either a direct estimation of turbulence response or ‘derivative’ coefficients, which enable more sophisticated numerical analysis of turbulence response to be carried out. The criteria for establishing the need to undertake these tests are given in BD 49/01.

Finally, tests may be required to simulate the salient properties of the wind and the characteristics of the environment of the bridge. These tests would be carried out if there was some doubt over the applicability of the topography rules in this Manual and a more precise method of calculating the wind environment was sought.

For most studies in the first two categories it is necessary to use large scale models to accurately simulate the structure, deck furniture and, possibly, highway or railway traffic, and wind tunnels operating with uniform laminar flow (aeronautical wind tunnels) are used. More accurate measurements of mean loads require a simulation of the turbulence characteristics of wind, but this would require a model whose scale would be too small to be practicable. Smooth flow tests are thus generally acceptable for these measurements providing upper bound values to the coefficients when compared in the natural wind.

Wind tunnels designed to develop the type of flow associated with the third category are classified as boundary layer wind tunnels (BLWT). The required small scale of the topography is such that a realistic model of the bridge itself would be impracticable.

The information to be extracted from wind tunnel tests depends on the type of test undertaken. Table E1 provides a summary of the various types of test and lists both the input parameters for the test and the information that should be collected. The specific requirements are discussed in the following sections.

Methodologies and procedures by which the information can be interpreted are virtually impossible to 'codify' in the sense of providing guidance. This must depend on the expertise and experience of both the testing establishment and the engineer for whom the tests were undertaken.

Part of that expertise comes in recognising and investigating the effects of small changes to details on the bridge. Parametric studies of all significant variables will benefit the design greatly. The sensitivity of the deck to the both changes in the intensity of turbulence and angle of inclination of the wind will provide significant insight into the dynamic behaviour of the structure. Removing the deck furniture to assess its contribution to the overall structural response can also provide useful design information and enable the designer to seek further economy within the deck section without adversely affecting stability.

Examples of these parametric studies abound with the testing of the existing bridges in Hong Kong. The sensitivity of the shape of the splitter rail of the Tsing Ma bridge is well known to those involved in the testing; although it came to light as a result of the rigorous testing requirements of the Highways Department. Similarly the sensitivity of the narrow band response of the Ting Kau Bridge to increased turbulence was noted through detailed and thorough testing.

Thus, to undertake a meaningful interpretation of the wind tunnel results, the engineer must have recourse to results of similar bridge decks so as to be able to identify expected trends. The variation of the results with change of angle of inclination must be followed closely and any significant increase in rate of change of any parameter must also be investigated. The physical processes behind the increase need to be understood as well. The sensitivity of details that are likely to be affected by scaling effects must also be reviewed to determine if the results from the wind tunnel replicate actual full-scale response.

Table E1 – Input and Output Parameters for Wind Tunnel Testing

Type of test	Wind conditions used	Basic Information Required	
		Input	**Output**
Topographic models	Simulated turbulent flow with ground roughness and terrain modelled including significant local features (buildings, etc.)	a) Mean wind speed b) Variation of mean speed with height c) Terrain roughness upstream d) Simplistic rigid models of structures	i) Mean speeds at various locations both in plan and elevation ii) Inclination of mean wind speed to the horizontal iii) Locations of zones of accelerated flow iv) Turbulence intensities at specific locations v) Correlation of wind speeds
Local Environment	Simulated turbulent flow with all adjacent structures modelled	a) Mean wind speed b) Variation of mean speed with height c) Terrain roughness upstream d) Rigid models of elements	As for topographic models, but limited to zones around bridge structure under consideration. Typically wind flow around embankments, piers, towers, etc. (e.g. modelling of wind barriers)
Full Aeroelastic models	Simulated turbulent flow with ground roughness modelled	e) Mean wind speed f) Variation of mean speed with height g) Terrain roughness upstream h) Full dynamic model of bridge structure	i) Mean wind speeds, wind turbulence intensities and wind spectra at deck level and at other specified locations (e.g. top of towers for cable supported bridge) ii) Frequencies and descriptions of modes of vibration of bridge iii) Inherent damping of model iv) Mean forces on bridge deck in lateral, vertical and torsional directions v) Plots of response of bridge against wind speed at various specified locations (tower tops, mid span deck level) in terms of acceleration (in three directions) converted to amplitudes. vi) Overall drag and lift on structure through measurement of strains (e.g. in supports) vii) Incomplete bridge model to derive stability limits during erection
Section model tests for aerodynamic stability	Mean flow with simulated turbulence at specified intensity (but incorrect scale). Intensity to be carefully chosen to simulate full scale behaviour. Scale inertia forces correctly	a) Mean wind speed b) Turbulence intensity (if relevant) c) Wind incidence to horizontal d) Frequency ratio of mounting e) Range of damping values to be used f) Inclusion of modelled traffic	i) Critical wind speeds and responses (amplitudes and accelerations) for the onset of vortex shedding (flexural and torsional) vibrations ii) Critical wind speeds for the onset of galloping (flexural and torsional) instabilities. iii) Critical wind speeds for the onset of flutter (classical and stalling) instabilities. iv) Critical wind speeds for the onset of torsional divergence. v) Aerodynamic derivatives (Pi, Hi and Ai) using Scanlan's proposals for aerodynamic stability
Section model tests for static coefficients	Mean flow	a) Mean wind speed b) Wind incidence to horizontal c) Inclusion of modelled traffic	i) Static drag, lift and torsional coefficients both with and without traffic (if relevant) ii) Test with and without deck furniture of erection conditions are required

E.3 SECTIONAL MODEL TESTS

E.3.1 Determination of Time Average Coefficients

Tests on sectional models of bridge decks can be used to determine the mean or static components of the overall wind load on the model. These wind loads can be obtained using rigid models with geometrically scaled features, in smooth flow wind tunnels. The size of the model should not be so large as to create significant blockage effects in the tunnel; corrections can be made to the results for low blockage effects but care needs to be taken to account for this properly.

For sections comprising circular section members or other curved surfaces that are likely to be Reynolds number (R_e) sensitive, adjustments based on full-scale data and/or theoretical considerations may be necessary. Modelling adjustments are commonly needed for very small elements such as handrails, cables or pipes to avoid local R_e below about 500.

The effect of wind inclination in elevation should be examined. The extent of this effect should be judged on the site topography, any planned super-elevation of the bridge and predicted torsional deflections under traffic loads. Generally tests up to ±5° are adequate. BD 49/01 requires values of less than this and the guidance given therein is generally sufficient, however, it is prudent to assess the complete picture of structural sensitivity to inclined angles of incidence.

Clearly, for sectional models, only wind normal to the centreline of the bridge can be considered and wind inclined in plan cannot be modelled.

In sectional models isolated objects, such as cable anchorages, sign gantries and lighting columns need to be considered, usually in subsequent desk-top analyses, when assessing total drag, lift and twist on the bridge section.

Simulated traffic patterns can be modelled to determine the appropriate coefficients for the conditions of wind with traffic. Such models may represent high density highway traffic – simple block models of typical Heavy Goods Vehicles (HGVs) and light traffic are generally used – or railway or pedestrians, as relevant.

E.3.2 Determination of Overall Wind Loads

Accurate measurements of both the mean and the dynamic components of the overall loads can only be obtained if both the approach flow and the local environment are properly simulated. For the model to bridge scale required this becomes impracticable. Recourse therefore needs to be made to aeroelastic models (see E.4) or CFD approaches.

E.3.3 Section Model Tests to Determine Aerodynamic Stability

The primary objective of such tests is to determine the aerodynamic stability of the bridge deck, mounted with deck furniture, using a geometrically scaled model of a section of the bridge elastically mounted in a wind tunnel. Typically, such models simulate the lowest bending and torsional vibration frequencies and are tested in uniform laminar flow. If lateral frequencies are likely to couple with bending and/or torsion, allowance should also be made for providing appropriate horizontal elastic supports.

The requirements of geometric scaling and Reynold's number limitations, outlined in E.3.1 above, still apply. In more advanced or refined stages, section models are tested in simulated turbulent flow in order to provide estimates of the responses at sub-critical wind speeds. Clearly at the scale of R_e modelled, supercritical flow is not feasible. As the simulated turbulence generally has a preponderance of the smaller-size eddies most likely to influence flow features such as vortex-shedding or re-attachment, the total intensity of turbulence should be selected with care and generally should be significantly lower than the standard atmospheric value for full size. Depending on the wind spectra modelled, a wind tunnel model with 6% turbulence intensity could correspond to a full-scale intensity of over twice this value. Reliance on beneficial effects from turbulence must not be allowed to reduce the likely aerodynamic effects.

In addition to modelling the geometry in accordance with E.3.1 above, it is necessary to maintain a correct scaling of inertia forces, the time scale, the frequency and the structural damping. The time scale is normally set indirectly by maintaining the equality of the model and full-scale reduced velocities of particular modes of vibration. The reduced velocity is the ratio of a reference wind speed and the product of a characteristic length and the relevant frequency of vibration. For example, the reduced velocity for flutter may be expressed as:

$$\frac{V_f}{f_T b}$$

where V_f is the critical wind speed for flutter;
f_T is the torsional frequency in the mode under consideration; and
b is normally taken as the total width of the bridge.

Measurements should be carried out through the range of wind speeds likely to occur at the site to provide information on both relatively common events, which influence serviceability, and relatively rare events, which govern ultimate strength behaviour. Wind inclination in elevation should also be examined.

Measurements of vortex excitation require careful control of the wind speed around the critical velocity and care must be exercised if divergent amplitudes are predicted, to ensure that these do not become so violent as to destroy the model.

E.3.4 Aerodynamic Derivatives

Increasing use has been made of aerodynamic derivatives for the determination of critical wind velocities for bridge decks. This methodology was pioneered by Scanlan and has gained considerable acceptance by bridge engineers (Kap Shui Mun Bridge was tested in the fashion). There are several methods of undertaking tests to obtain the aerodynamic derivatives. These include :

(a) Vibration tests where the deck is given an initial vertical and torsional displacement. The aerodynamic derivatives may be calculated by considering the transient behaviour of the deck under free vibration at various wind speeds in the wind tunnel.

(b) Forced oscillation tests where the motion of the model is forced in a predetermined fashion, with the aerodynamic forces being measured directly from the surface of the model.

It has been noted by Hansen[3] that these two methods produced very different results for the Storebælt Bridge. The first method is the more generally used method and it is postulated that the differences in results are a function of the difficulties of obtaining sufficient measurements in the second method. Consequently, it will be important to ensure that if these methods of wind tunnel testing are to be used; testing should only be undertaken by wind tunnel experts who have a demonstrated track record in these methodologies.

E.4 AEROELASTIC SIMULATIONS OF BRIDGES

Ideally a dynamic model of the full bridge should be used in the wind tunnel, commonly referred to as an aeroelastic model, to provide information on the overall wind induced mean and/or dynamic loads and responses of bridges. In such cases the model is built to correctly represent the stiffness, mass and damping properties of the structural system and the aerodynamically significant features of the bridge's geometry. Such models are particularly valuable for slender, flexible and dynamically sensitive structures, where dynamic response effects may be significant. However to be representative, such tests must also consistently model the salient characteristics of natural wind at the site. It is only possible to model the full spectrum of atmospheric turbulence in a wind tunnel at small scale; together with the obvious constraint of fitting a full bridge model within the tunnel, this is generally irreconcilable with the scale desirable to ensure correct behaviour, which is commonly sensitive to small changes in cross-section. For this reason the primary study should be made by section model tests; where non-uniformity of section or of incident flow conditions, complex dynamics or erection considerations, necessitate the use of a full model, particular care is needed in its design and interpretation.

As the modelling of dynamic properties requires the simulation of the inertia, stiffness and damping characteristics of only those modes of vibration that are susceptible to wind excitation, approximate or partial models of the structural system are often sufficiently accurate.

Aeroelastic models may also be used to investigate bridges under construction. Certain bridge types, particularly box-girder suspension bridges, will have lower critical flutter speeds for the partially erected state. Data presented by Tanaka[5] shows the variation in critical wind speed at various stages of construction of several large suspension bridges. The magnitude of the variation is dependent on the method of construction (centre-span inwards or from the towers outwards), but shows that the greatest risk occurs with about 10 to 20% of the deck erected.

E.5 STUDIES OF THE WIND ENVIRONMENT

E.5.1 Topographic Models

Information on the characteristics of the full-scale wind may not be available in situations of complex topography and/or terrain. Small scale topographic models, with scales in the range of 1:2000, can be used in such situations to provide estimates of the subsequent modelling of the wind at a larger scale, suitable for studying particular wind effects on the bridge.

E.5.2 Local Environment

Nearby buildings, structures and topographic features of significant relative size influence the local wind flow and hence must be allowed for in simulations of wind at particular locations. For bridges in urban settings this requires the scaled reproduction (usually in block outline form) of all major buildings and structures within about 500 m to 800 m of the site. Also of particular importance is the inclusion of major nearby existing and projected buildings which could lead to aerodynamic interference effects, even though they may be outside this 'proximity' model.

Corrections are generally required if the blockage of the wind tunnel test section by the model and its immediate surroundings exceeds about 5 to 10%. Typical geometrical scales used in studies of overall wind effects or for local environment tests range between about 1:300 to 1:600.

E.5.3 Use of BLWTs

A BLWT should be capable of developing flows representative of natural wind over different types of full-scale terrain. The most basic requirements are as follows :

- to model the vertical distribution of the mean wind speed and the intensity of the longitudinal turbulence; and

- to reproduce the entire atmospheric boundary layer thickness, or the atmospheric surface layer thickness, and integral scale of the longitudinal turbulence component to approximately the same scale as that of the modelled topography.

In some situations a more complete simulation including the detailed modelling of the intensity of the vertical components of turbulence becomes necessary.

E.6 INSTRUMENTATION

The instrumentation used in wind tunnel model tests of all aforementioned wind effects should be capable of providing adequate measures of the mean and, where necessary, the dynamic or time-varying response over periods of time corresponding to about 1 hour in full-scale. In the case of measurements of wind induced dynamic effects, overall wind loads and the aeroelastic response, the frequency of the instrumentation system should be sufficiently high to permit meaningful measurements at all relevant frequencies and avoid magnitude and phase distortions.

Furthermore, all measurements should be free of significant acoustic effects, electrical noise, mechanical vibration and spurious pressure fluctuations, including fluctuations of the ambient pressure within the wind tunnel caused by the operation of the fan, opening of doors and the action of atmospheric wind. Where necessary, corrections should be made for temperature drift.

Most current instrumentation systems are highly complex and include on-line data acquisition capabilities which, in some situations, are organised around a computer which also controls the experiment. Nevertheless, in some situations it is still possible to provide useful information with more traditional techniques including smoke flow visualisation. Although difficult to perform in turbulent flow without proper photographic techniques, flow visualisation remains a valuable tool for evaluating the overall flow regime and, in some situations, on the potential presence of particular aerodynamic loading mechanisms.

E.7 QUALITY ASSURANCE

The reliability of all wind tunnel data must be established and should include considerations of both the accuracy of the overall simulation and the accuracy and hence the repeatability of the measurements. Checks should be devised where possible to assure the reliability of the results. These should include basic checking routines of the instrumentation including its calibration, the repeatability of particular measurements and, where possible, comparisons with similar data obtained by different methods. For example, mean overall force and/or aeroelastic measurements can be compared with the integration of mean local pressures.

Ultimate comparisons and assurances of data quality can be made in situations where full-scale results are available. Such comparisons are not without difficulties as both the model and full-scale processes are stochastic. It is also valuable to make credibility crosschecks with the code requirements and previous experience.

E.8 PREDICTION OF FULL-SCALE BEHAVIOUR

The objective of all wind tunnel simulations is to provide direct or indirect information on wind effects during particular wind conditions. The requirements for full-scale predictions should be clearly defined in the performance specification given to the wind tunnel testing facility.

Information that will be required includes the full-scale critical velocities for the onset of aeroelastic instabilities. Typically, wind tunnel results are presented in terms of reduced velocities; the conversion to full-scale velocities is relatively simple but should be left to the wind tunnel facility to ensure the correct scale factors are used.

Static coefficients need to be defined in terms of a length scale. This would usually relate to the bridge depth or width, but care is required if the former is used. It needs to be stated if the depth of the deck includes the depth of any edge barriers. The predictions for the full-scale amplitudes for vortex excitation will need to be stated at either peak or root mean square values. Finally, if measurements have been undertaken in turbulent flow, the intensity of turbulence needs to be reported for both the reduced and full-scale intensities. This is particularly important for checking the validity of vortex-induced response predictions.

E.9 TYPICAL SCALES

One of the difficulties of wind tunnel testing is replicating the full-scale effects of the wind at the reduced scale available within the wind tunnel. It generally holds that the larger the scale of the model, the less likely additional errors will be introduced from inaccuracies in the model making. Table E2 gives typical scales that are used in wind tunnel tests for bridges. The scales given are the desired scale which, based on experience, provide results that are within the bounds of accurate model making.

Testing at larger scales is likely to be beneficial and should be encouraged. Conversely, testing at smaller scales should be avoided. However, the values given below are not absolute limits and may be exceeded providing the testing is carried out at a reputable facility, with a proven international track record.

Table E2 – Typical Scales for Wind Tunnel Testing

Type of Test	**Typical Scale**
Topographic models	1:2000
Local environment	1:600 to 1:300
Aeroelastic models	1:200 to 1:100
Section models (stability or time average coefficients)	1:80 to 1:40
Models of ancillaries	> 1:20

E.10 REFERENCES

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