Land Transport Authority

ENGINEERING GROUP

CIVIL DESIGN CRITERIA
FOR ROAD AND RAIL TRANSIT SYSTEMS

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Controlled Document

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GENERAL

1.1 INTRODUCTION

1.1.1 Scope

The Design Criteria give the requirements for the design and detailing of all Civil Engineering Works for the Land Transport Authority.

1.1.2 Definitions

1.1.2.1 The definitions of “Authority”, “Contractor” and “Works” etc. shall be those given in the Conditions of Contract.

1.1.2.2 The term Engineer used in the Design Criteria refers to the Engineer appointed by the Authority for the purposes of the Contract. Where the Conditions of Contract require instead that a Superintending Officer be appointed for the purposes of the Contract, the term Engineer in this Specification shall refer to the Superintending Officer so appointed by the Authority.

1.1.2.3 The use of the terms “rapid transit”, “stations” etc, shall be taken to apply to all guided systems, whether MRT or LRT, whether steel on steel or rubber tyres on guideways, unless specifically stated otherwise or agreed otherwise with the Engineer.

1.1.3 General Obligations

1.1.3.1 Compliance with Statutory Requirements and International Standards.

1.1.3.1.1 All designs shall be carried out and fully endorsed by Professional Engineers holding a valid practising certificate and registered under the Professional Engineers Act, Singapore in the civil and/or structural discipline, and also by registered Accredited Checkers in accordance with the Building Control Act.

1.1.3.1.2 All designs shall comply with relevant regulations, Building Control Act and Fire Safety Act.

1.1.3.1.3 Compliance with a Singapore Standard (SS)/ Singapore’s adoption of Eurocode (SS EN) or British Standard (BS)/ British’s adoption of Eurocode (BS EN) or a standard approved by the Authority (or accepted by the Engineer) or the requirements of these Design Criteria shall not confer immunity from legal obligations.

1.1.3.2 Adjacent Works

The design shall take into account any constraints or effects imposed by the existing and planned works and services in the surrounding areas, and works of other nearby developments.

1.2 STANDARDS

1.2.1 The design of all Works shall comply with the appropriate current standards with its Annexes and/or Codes of Practice issued by Enterprise Singapore, or if such a standard and/or Code of Practice does not exist, then the appropriate current standard with its Annex issued by the British Standards Institution (BSI) shall apply.

1.2.2 If no specific guidance is given in Eurocodes, the designer shall propose a list of non-contradictory complementary information (NCCI) including relevant Design Manual for Roads and Bridges (DMRB)/ BSI Published Documents to be used where appropriate and subjected to written approval by the Engineer.

1.2.3 If an appropriate standard from Enterprise Singapore and BSI does not exist and no other standard is stated in the Contract Documents, then subject to the acceptance of the Engineer and the Commissioner of Building Control of the Building and Construction Authority, an appropriate current standard from a reputable institution may be used. Three English language copies of such proposed standards shall be submitted to the Engineer.

1.2.4 Generally, the requirements spelt out in the Particular Specification, General Specification, M&W Specification and the Design Criteria shall take precedence over any relevant Singapore or British Standards and Annexes, Highways England's standards and advisory notes or other International Codes of Practices.

1.2.5 Where metric unit and imperial unit version of the same standard exist, the metric version shall apply.

1.3 DESIGN

1.3.1 Design Objectives

1.3.1.1 The design of structures and civil engineering works shall meet the following objectives: they shall be safe, robust, economical, durable, with operation and maintenance costs reduced to a practicable minimum, and shall be fit for purpose. Simplicity of structural form and layout is preferred. All structures shall be designed to be aesthetically pleasing.

1.3.1.2 The elements of all structures shall be designed and detailed to achieve the design objectives by, inter alia, the following:

a) appropriate selection of materials
b) consideration of the long term deterioration of materials in the service environment
c) due care in design and detailing so as to facilitate good workmanship in construction and the achievement of design intent
d) consideration of access and other requirements for inspection and maintenance
e) adoption of good engineering practice
f) use of safe construction methods and proven techniques

1.3.1.3 The durability objective of the project shall be to achieve a service life, with appropriate maintenance, of 120 years for all permanent structures. The measure of achievement of durability shall be that all the criteria set in the design shall be maintained throughout the service life. Deterioration of materials shall be taken into account in the design and specification of the works.

1.3.1.4 Due diligence and skills shall be applied in the design and detailing to ensure that the works can be constructed economically, practically and safely.

1.3.1.5 All structural designs shall comply with all the ultimate and serviceability limit states.

1.3.2 Design of Temporary Works

1.3.2.1 All Temporary Works shall be designed and detailed to be compatible with the Permanent Works, in accordance with Building Control Act and Regulations, and Workplace Safety and Health (WSH) Act and Regulations.

1.3.2.2 Temporary Works designs shall be carried out and endorsed by a Professional Engineer.

1.3.3 Oversite and Adjacent Developments

All structures are to be designed wholly independently of any benefit which might be obtained from oversite or adjacent development. For example, in consideration of stability against flotation or of any lateral loading, the design should allow for the development not being present if that gives a more onerous design case.

1.4 DESIGN CALCULATIONS

1.4.1 Method of Calculations

1.4.1.1 Unless otherwise varied by the subsequent Chapters of the Design Criteria, all calculations shall be carried out in accordance with the requirements and recommendations of appropriate current Standards.

1.4.1.2 The use of "State-of-the-Art" methods of calculations or methods that have not been extensively tried and proven within the industry will not be permitted unless prior acceptance for their use has been obtained from the Engineer. The design shall be in accordance with established good engineering practice and principles.

1.4.2 Use of Engineering Software

1.4.2.1 All software to be used shall be accepted by the Engineer.

1.4.2.2 The software to be used shall be those that are produced by reputable software houses, have undergone extensive testing and have been successfully used and proven in similar projects. In this respect, the relevant documents and sample calculations to demonstrate the accuracy and reliability of the software shall be submitted. Details of computer software, including assumptions, limitations and the like, shall be clearly explained in the design statement.

1.4.2.3 All input and output data of the engineering software shall be clearly defined and the calculations shall include clear and unambiguous information of what each parameter means in the computer output.

1.4.2.4 When in-house spreadsheets are used, the proposed version of the spreadsheet shall be clearly indicated and submitted together with hand calculations to verify the results of the spreadsheet for all possible calculation scenarios. A print-out of the spreadsheet showing the formulae normally hidden shall also be submitted with the cell references clearly labelled along the top and left hand margins of each page.

1.4.3 SI Units

All calculations shall be carried out and presented in SI Units as specified in BS EN ISO 80000-1. The units of stress shall be N/mm^2 or kN/m^2.

1.4.4 Language

All calculations and other documents shall be submitted in the English Language.

1.5 SURVEY & SETTING OUT

1.5.1 Levels

All levels given on the design drawings shall refer to Singapore Height Datum (SHD).

1.5.2 Co-ordinates

All co-ordinates given on the design drawings shall be based on the project co-ordinate system as defined in the Particular Specification. The project co-ordinate system shall be clearly defined and indicated on the design drawings.

1.6 DURABILITY ASSURANCE

1.6.1 Design Considerations

1.6.1.1 The design shall address the durability of all elements of the structures. The design process shall incorporate an assessment of potential deterioration of materials in their exposure environments (e.g. exposure to ground water) throughout the service life, including but not limited to:

a) durability of concrete,

b) corrosion of metals,

c) long term performance of sealants, waterproofing, coatings and other forms of protection.

1.6.1.2 Construction processes, which are critical to the achievement of durability, shall be identified. These include workability requirements for casting concrete around relatively congested reinforcement sections, and duration of placement in terms of delay in setting to avoid cold joints.

1.6.2 Critical Elements

Particular attention shall be given to deterioration of elements, which cannot practically be accessed for maintenance or repair during the service life. In the case of such critical elements, the design shall be premised on the element (including all its components) remaining durable throughout the service life without maintenance. Additional measures shall be incorporated in the design of such elements to address the eventuality of the primary protection failing to achieve the desired durability. Where normal methods of inspection are impossible, provision for monitoring material performance by instrumentation shall be implemented where practicable.

1.6.3 Durability Assessment

1.6.3.1 Based on the durability objectives of the project, performance criteria for materials shall be developed from an assessment of the following: -

a) the micro-environment to which the element is exposed
b) potential deterioration mechanisms in this micro-environment
c) the likely material life
d) the feasibility and cost of in situ monitoring, maintenance and/or repair
e) the necessity and cost-effectiveness of providing additional protection
f) the significance of deterioration

1.6.3.2 In addition to the assessment of "the likely material life", the quality control tests to monitor the quality of ready-mix concrete for durability and the acceptance criteria shall also be provided.

1.6.3.3 Any proposal to revise the Materials and Workmanship Specifications shall be based on the performance criteria arising from such considerations.

1.6.4 Life Cycle Cost Analysis

1.6.4.1 Where required, life cycle cost analysis shall be undertaken as a basis for selection of materials. Such analysis will require establishment of material performance and life of all components of the element and compare the total life cycle costs of viable options, by summation of: -

a) initial capital cost;
b) total recurrent costs of inspection, monitoring, maintenance/repair; and
c) replacement cost.

1.6.4.2 Total life cycle costs, shall be expressed in net present value by using discounted cash flow techniques based on 3.4% discount rate or agreed value. The analysis is to be used as a decision making process, hence the life cycle costs need only be sufficiently accurate for the purpose of comparison of options. A sensitivity analysis shall be undertaken to reflect the uncertainties related to: -

a) predictions of material performance
b) workmanship in construction
c) unit rates for calculation of inspection, maintenance, repair and replacement costs.

1.6.5 The design characteristic strength, the maximum nominal aggregate size, the minimum cement content, the maximum cement content, and maximum free water: cement ratio and permitted cement types shall be shown clearly on the design drawings for reinforced, precast and prestressed concrete works together with any other restrictions on materials or properties required.

1.6.6 Drawings

1.6.6.1 The preparation of drawings shall comply with Drawing and CAD Standard (Microstation) and SS CP83.

1.6.6.2 Building Information Modeling (BIM) and BIM models shall be in compliance to Singapore BIM Guide and LTA BIM Requirements specified for the project.

1.7 MATERIALS AND WORKMANSHIP SPECIFICATION

Attention is drawn to the obligation to review the Materials and Workmanship Specification. The Materials and Workmanship Specification should however be regarded as a minimum standard. Any provision of the Materials and Workmanship Specification which appears incompatible with the design basis shall be highlighted, and appropriate modifications to the Materials and Workmanship Specification shall be proposed, and agreed with the Engineer.

1.8 DIMENSIONS

All dimensions given on the Authority's Drawings or within the Authority's documentation shall be taken to be minimum dimensions to be achieved on site after allowance for all construction tolerances, deflection of embedded walls, sagging of beams and floors, etc.

1.9 BLINDING

Reinforced and/or prestressed concrete shall be cast against an adequate concrete blinding and not directly against the ground. For the blinding concrete, the minimum concrete strength class and thickness shall be C12/15 and 50mm respectively. The thickness and strength of blinding may need to be increased depending on the softness and irregularity of the ground and the thickness of the concrete pour. Where the ground beneath the blinding is to be removed at a later date (for example in top-down construction) a debonding membrane shall be used at the interface between the blinding and reinforced concrete. The blinding and membrane details shall be indicated on the design drawings.

1.10 LAND BOUNDARIES

1.10.1 Definition of Land

1.10.1.1 State land are land owned by State.

1.10.1.2 Statutory Board land are land owned by Statutory Board only.

1.10.1.3 Non-State land are land owned by private individual, private company, trustees and Statutory Board land with tenant/lessee.

1.10.2 In determining the design of the Works, the designer shall take into account the land available to the Authority and the need to optimise land use. Existing land boundaries shall be observed to ensure no encroachment on adjacent properties.

1.10.3 For railway projects, the Authority takes legal ownership of that portion of State land occupied by station boxes, sub-stations, depots and ancillary facilities as determined by the Singapore Land Authority. Such State land shall include those parcels of non-State land which have been identified to be acquired for the railway project. For all other railway-related facilities which includes ventilation shafts, cooling towers, emergency escapes and entrance structures, on State land and Statutory Board land, the Authority exercises statutory rights under the Rapid Transit Systems Act

1.10.4 All rooms and revenue generating elements not essential to the operation of the railway (e.g. toilets, shops, kiosks, advertising panels and Civil Defence facilities) shall be positioned within the station box.

1.10.5 The Rapid Transit Systems Act only allows the permanent placement of railway tunnels and viaducts on non-State land. No other railway structures and facilities are allowed on non-State land. Where in the opinion of the Authority, the design and positioning of a railway related structure or facility is such that the Authority will be required to take legal ownership of the affected non-State land, the Authority reserves the right to reject such a proposal.

1.10.6 For design of road and commuter facility, the designer shall take into account that the roads are only built on State Land. Such State Land shall include those parcels of non-State land which have been identified to be acquired for the road project as well as those which had been set aside for road purposes as stipulated under a related planning condition.

1.10.7 In addition, the Street Works Act allows for the permanent placement of road structures and road related facilities on Statutory Board land. The road structures and road related facilities should be located on State land and within road reserve as far as possible. Road related facilities such as the installation of shelter or cover of either a pedestrian linkway or pedestrian overhead bridge are allowed to be located on Statutory Board land only as a matter of last resort and subject to the approval of the Authority and should be accessible from State land.

1.10.8 The layout of the Works shall take into account proposed and existing land boundaries to make full utilisation of available land. Excision of land parcels leading to creation of remnant unusable plots shall be avoided.

1.10.9 All facilities (for example lighting posts, handrails, inspection chambers, utility service meters, etc.) that serve the Road Works shall be sited within the road reserves.

1.10.10 All site layout plans, including those of working sites and casting yards shall show existing cadastral information, Road Reserves Lines, Railway Protection Zones and Drainage Reserves. The designer is advised that this information available from government agencies shall be verified with the precise survey controls established for the construction of the project. Due allowance in the form of specific field surveys to resolve critical differences shall be made in site layout design.

1.11 FLOOD PROTECTION

1.11.1 The Design Flood Level shall be in accordance with the prevailing requirements of Public Utilities Board (PUB) as defined in the Code of Practice on Surface Water Drainage.

1.11.2 All entrances, vent shafts openings, tunnel portals, service entries and other openings into underground railway structures and all road thresholds and perimeters to depressed carriageways, underpasses and road tunnels shall not be lower than the Design Flood Level.

1.11.3 Where drainage or sewerage pipes discharge from the underground structure into the surface system, swan necks shall be provided at a level above the Design Flood Level. If gravity drainage provisions are made, the drainage exit points shall be above the Design Flood Level to prevent any back flow of water into the sub-surface structures during floods.

1.11.4 Platform and Crest Level

Where entrances, vent shafts, tunnel portals or other openings into underground railway structures are located on a platform, the platform level shall be set at or above the Design Flood Level. The crest level (threshold level) of any opening on the platform shall be at least 300mm higher (crest level) than the platform level to prevent flooding from sudden downpours.

1.11.5 Adjoining Developments

The threshold level of any entrances and opening into any development with a connection to an underground station shall not be lower than the Design Flood Level of the station for the flood prevention of the rapid transit system. Any platforms at the entrances to adjoining developments shall also comply with the platform and crest level requirements above.

1.11.6 All arrangements for flood protection shall meet the requirements of PUB (Drainage).

CHAPTER 2

TRACK ALIGNMENT

2.1 SCOPE

2.1.1 The design of the track alignment shall comply with the functional requirements stipulated in this chapter and shall take full account of all relevant factors including the design criteria, requirements of Operation, Signalling, Traction power, Rolling stock, Track maintenance, Construction constraints and cost, Existing structures and utilities, Geotechnical and tunnelling conditions, Environmental impact and Land use.

2.1.2 The track alignment design shall ensure there is a continuity of traction power to prevent train stalling along the main line and depot due to loss of traction supply resulting from third rail gap.

2.1.3 The design shall be co-ordinated with all relevant designers, contractors and other authorities. The design shall comply with the specified desirable value. Values in the minimum or maximum range shall only be used where it is impossible to use the desirable value. The use of absolute minimum or maximum value must be demonstrated and is subject to the Engineer's acceptance.

2.1.4 The design criteria apply to the track alignment design for Mass Rapid Transit (MRT) system and it is a steel wheel running on steel rail system.

2.2 HORIZONTAL ALIGNMENT

2.2.1 Horizontal Curves

2.2.1.1 *Track gauge* is the distance measured between the inside face of the two running rails at a point 14mm below the top of the rails *(gauge points)*. For MRT rail systems track gauge shall be 1435mm.

2.2.1.2 *Horizontal alignment – non-tunnel* is the alignment based on a point midway between gauge points.

2.2.1.3 *Horizontal alignment – in tunnel* is the alignment based on a point on the track centreline at a height 1600mm above the rail line.

2.2.1.4 *Circular Curve* is a curve of constant radius.

2.2.1.5 *Compound Curve* is a curve formed of two or more circular curves of differing radii curving in the same direction. The circular curves may be linked by transition curves.

2.2.1.6 *Reverse Curve* is a curve formed of two or more circular curves curving in alternate directions which may be linked by transition curves. A reverse curve has no straight track between each circular curve but has abutting transition curves. For the purpose of the alignment, each part of a reverse curve shall be given a separate curve number.

2.2.1.7 The limits for radii for horizontal circular curves are shown below:

	Main Line		**Depot**	
	Desirable Minimum	**Minimum**	**Minimum**	**Absolute Minimum**
MRT system (aboveground)	500 m	400 m	190 m	140 m
MRT system (underground)	400 m	300 m	190 m	140 m

The absolute minimum value in depot is not applicable to the following items: -

- North South East West Line (NSEWL)
- North East Line
- Reception Track

The use of absolute value shall be subjected to the approval of the Engineer.

2.2.1.8 The track shall preferably be straight throughout the length of stations. The presence of external constraints may necessitate limited encroachment of curves at station ends.

2.2.1.9 Track through platforms shall be straight. The start of a horizontal curve, transition curve or a turnout shall have a minimum distance of 20m from the platform end to avoid the vehicle throw affecting the platform nosing clearance. Where encroachment is unavoidable, this shall be limited such that the combined effects of vehicle throw and cant do not affect the location of the nosing at platform ends by more than 20mm when compared to straight track.

2.2.1.10 Circular curve radii shall be selected to be the maximum practicable. The radius selected for any particular curve shall not be so large as to unnecessarily impose more severe curvature of the track at either end of that curve.

2.2.1.11 The length of elements for circular curve or tangent shall be:

Main Lines	
Desirable (m)	≥ 0.40 x V
Minimum (m)	≥ 0.25 x V
Absolute Minimum (m)	≥ 0.15 x V or 20m (whichever is larger)
Non-revenue Lines	
Minimum	7m

Where V = Operating Speed (km/h)

2.2.1.12 The use of compound curves and reverse curves without straight element shall be avoided wherever possible.

2.2.2 Cant and Speed

2.2.2.1 *Cant,* ***D*** *(Superelevation)* is the vertical distance by which one rail is raised above the other and measured between the crowns of the two running rails. Cant is positive when the outer rail on a curve is above the inner rail or negative when it is below the inner rail.

2.2.2.2 *Equilibrium Cant,* D_{EQ} is the cant required to enable a vehicle to navigate a curve at a particular speed in order to balance centrifugal forces.

2.2.2.3 *Applied Cant* is the actual cant specified for the curve.

2.2.2.4 *Cant Deficiency,* ***I*** is the amount by which the applied cant is less than the equilibrium cant for the speed being considered.

2.2.2.5 *Excess Cant,* ***E*** is the amount by which the applied cant is greater than the equilibrium cant for the speed being considered.

2.2.2.6 *Cant Gradient,* ***dD/ds*** expressed as a dimensionless ratio, is the gradient at which cant is increased or reduced.

2.2.2.7 *Rate of Change of Cant,* ***dD/dt*** *or of Cant Deficiency,* ***dI/dt*** in millimetres per second is the rate at which cant or cant deficiency is increased or reduced relative to the speed of the vehicle.

2.2.2.8 *Maximum permissible speed* is the highest speed at which a train may traverse a curve with associated transition curves when radius, cant, cant deficiency, cant gradient and rates of change of cant and cant deficiency have all been taken into consideration.

2.2.2.9 *Restricted Speed* is the nominal maximum permissible speed for a section of track imposed by means of a *permanent speed restriction* and is determined by track geometry.

2.2.2.10 *Operating Speed* at a particular point on the track is the speed of the train at that point calculated from the coasting run speed profiles running under Automatic Mode prepared by the signalling designer.

2.2.2.11 The *curve*-speed-cant relationship shall be based on the following equations: -

$$D_{EQ} = (D + I) = \frac{11.82\ V^2}{R}$$

Where,

D_{EQ} = Equilibrium cant (mm)

D = Applied Cant (mm)

I = Cant deficiency (mm)

R = Horizontal curve radius (m)

V = Speed (km/h)

Equilibrium cant is to be rounded and used without decimals

Applied cant is to be rounded up in 5mm intervals

Formula is only applicable for a track gauge of 1435mm.

2.2.2.12 The maximum allowable applied cant shall be: -

	Desirable Maximum	Absolute Maximum
For concrete track	125 mm	150 mm
For ballasted track	110 mm	125 mm

2.2.2.13 The amount of cant deficiency or non-compensated lateral acceleration shall be limited to the following: -

	Cant Deficiency	**Lateral Acceleration**
Plain Line		
Desirable Minimum	25 mm	0.16 m/s^2
Desirable Maximum	100 mm	0.65 m/s^2
Absolute Maximum	110 mm	0.72 m/s^2
Turnout		
Desirable Maximum	80mm	0.523 m/s^2
Absolute Maximum	100mm	0.654 m/s^2

The maximum non-compensated lateral acceleration in the running plane

$$a_q = \frac{I \times g}{s}$$

Where,

a_q = Non-compensated lateral acceleration in the running plane (m/s^2)

g = acceleration due to gravity (9.81m/s^2)

I = cant deficiency (mm)

s = track centres (1500mm)

2.2.2.14 The desirable cant value shall be selected as:

Applied cant, $D = D_{EQ} - 25$

Where,

Cant deficiency $I \geq 25$mm is selected.

Cant of less than 25mm need not be applied.

Cant shall be selected to suit the operating speed. Consideration for both cant and cant deficiency shall also take into account the requirements of cant gradient, rate of change of cant and cant deficiency.

2.2.2.15 Where constraints on the alignment design are such that the requirements of Clause 2.2.2.14 cannot be met, a permanent speed restriction shall be imposed. Such restrictions shall be minimised as far as practicable.

2.2.2.16 In cases where the operating speed of the train on part or all of a curve is considerably less than the maximum permissible speed, it may be necessary to impose a permanent speed restriction to ensure that any excess cant at the design speed is kept to a practical minimum.

2.2.2.17 Application of cant must be introduced throughout the length of a transition curve. The only exception is the back to back transition curve, where the cant is introduced over the entire length of both transition curves.

2.2.3 Transition Curves

2.2.3.1 *Transition Curve* is a curve of progressively varying inverse radius (curvature = 1 / R) used to link either a straight with a circular curve, or two circular curves of different radii.

2.2.3.2 *Shift* is the amount by which the centre of radius of a circular curve needs to move due to the placement of transition curves.

For estimation purposes: -

$$\mathrm{Shift} \approx \frac{\mathrm{L}^2}{24 \times \mathrm{R}}$$

Where,

L = Length of transition curve (m)

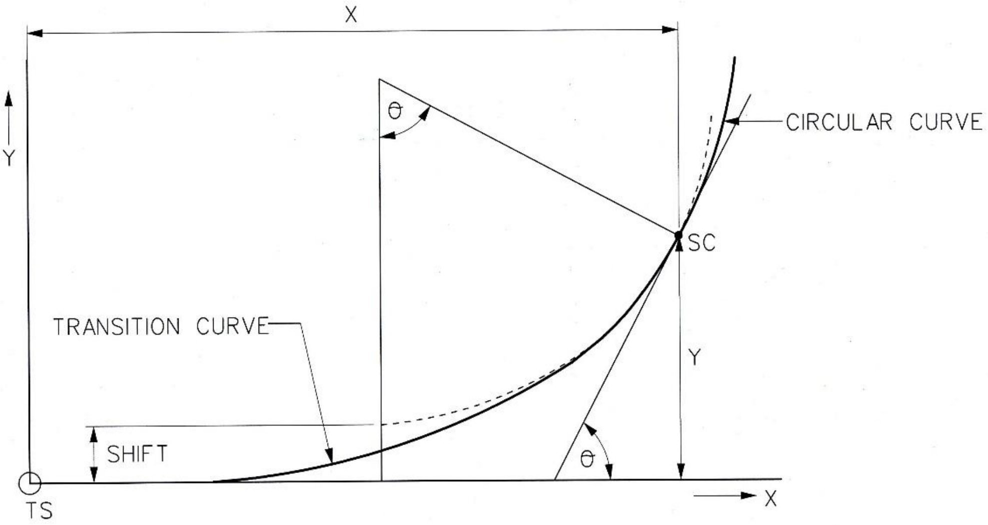
R = Radius of circular curve (m)

Transition curve can be omitted where the shift is less than 10mm.

2.2.3.3 In general, for all main line tracks, transition curves shall be provided at every changing point of alignment if the cant deficiency becomes $I \geq 30$mm or $Ea_1 \neq Ea_2$ of adjacent elements.

2.2.3.4 Transition curves shall be clothoids. The formula for clothoid is as shown in the following Figure 2.1: -

Figure 2.1 – Clothoid Transition Curve



$$\theta = \frac{L}{2R}$$

$$a = \left(\frac{d}{L}\right)^2 \theta$$

$$X = d\left[1 - \frac{a^2}{10} + \frac{a^4}{216} - \frac{a^6}{9{,}360} + \frac{a^8}{685{,}440}\right]$$

$$Y = d\left[\frac{a}{3} - \frac{a^3}{42} + \frac{a^5}{1{,}320} - \frac{a^7}{75{,}600} + \frac{a^9}{6{,}894{,}720}\right]$$

Where,

R = horizontal curve radius (m)

TS = point of change from tangent to transition curve

SC = point of change from transition curve to circular curve

L = overall (TS to SC) transition curve length (m)

d = transition curve length from TS to any point on transition curve

θ = central angle of transition curve

X = tangent distance of any point on transition curve with reference to TS

Y = tangent offset of any point on transition curve with reference to TS

2.2.3.5 The cant gradient ***dD/ds*** shall be subject to the following limits: -

Absolute maximum = 1 : 500

Desirable = 1 : 8 x V (the equivalent of 35mm/s)

Minimum = 1 : 3000 (due to constructability)

Where V = Operating Speed (km/h)

2.2.3.6 The rate of change of cant ***dD/dt*** shall be limited as follows: -

Plain Line	
Desirable Maximum	35 mm/s
Absolute Maximum	50 mm/s

The rate of change of cant deficiency ***dI/dt*** shall be limited as follows: -

Plain Line	
Desirable Maximum	35 mm/s
Absolute Maximum	55 mm/s

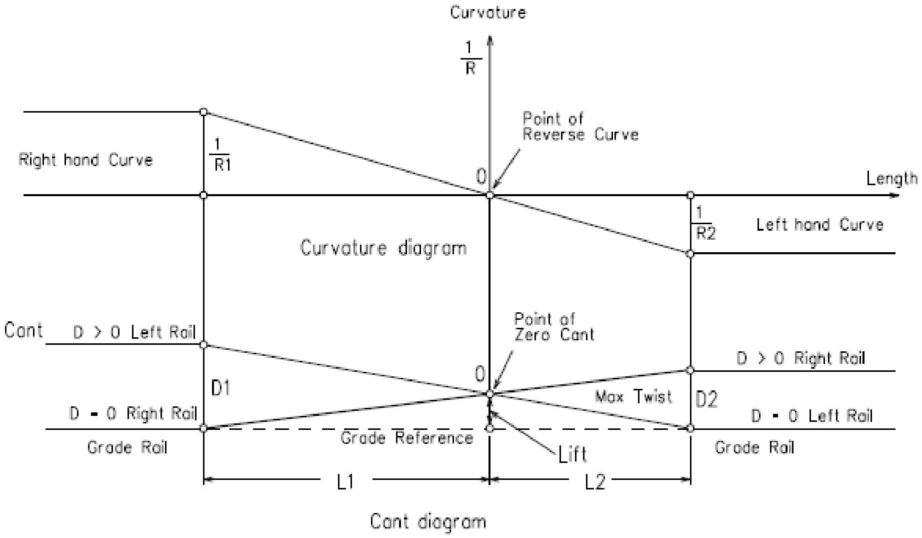
2.2.3.7 The limiting length of transition curve (Lt) are as shown below:

Line	Lt (m)
Desirable	$8 \times \mathrm{V} \times \mathrm{D}$
	$\frac{V \times D}{35 \times 3.6}$
	$\frac{\mathrm{V} \times \mathrm{I}}{35 \times 3.6}$
Minimum	$5 \times \mathrm{V} \times \mathrm{D}$
	$\frac{\mathrm{V} \times \mathrm{D}}{50 \times 3.6}$
	$\frac{\mathrm{V} \times \mathrm{I}}{55 \times 3.6}$
Absolute Minimum	20
	$5 \times \mathrm{V} \times \mathrm{D}$

Tracks within depot boundary do not require transition curves length due to low speed with the exception of the test track.

2.2.3.8 Where minimum tangent distance is not feasible between reverse curves, a back to back transition curves should be employed. The point of zero cant shall coincide with the point of reverse curve.

Figure 2.2 –Cant & Curvature Diagram of Reverse Transition Curves



The difference of both cant gradients must not exceed 1:500. The length of transition curves may differ but follow the same rule: -

$$\frac{D1}{L1} = \frac{D2}{L2}$$

Where,

L1 = first length of transition curve

L2 = second length of transition curve

The grade rail is the connection of non-elevated inner rails.

Additional design consideration must be investigated in detail before incorporation into the design including the lift in rail level at the point of reverse curve and increased clearance requirement.

2.2.4 Chainages

2.2.4.1 The datum of chainages for new lines shall be defined and approved by the Engineer.

2.2.4.2 Chainages shall be quoted in metres to three decimal places and shall be measured along the centreline of each individual track in plan with no correction for differences in elevation.

2.2.4.3 Initially a nominal 10m jump in chainage shall be provided on each track at each station centreline. Subsequent alignment revisions that results in changes to chainages shall be reflected by revising the jumps. The chainage at Contract boundaries shall not be changed. The starting chainage at each subsequent station centre for both bounds shall be identical.

2.2.5 Co-ordinates

2.2.5.1 Calculations for the setting out of the horizontal alignment for each track shall be based on co-ordinates of horizontal curve main points (begin and end of each alignment element) of the nominal track centreline.

2.2.5.2 Co-ordinates of all curve main points and radius points shall be clearly tabulated in metres correct to four decimal places.

Table must indicate: -

a) Speed (by geometry and permissible limit)

b) Chainage

c) Type (point abbreviation)

d) Radius

e) Length of element

f) Cant / cant deficiency

g) Rate of change of cant and cant deficiency

h) Cant gradient

The data should be listed in succession.

2.2.5.3 Horizontal curves radii shall be quoted in metres correct to three places of decimals and shall be the actual required radii after shift has been taken into account. Deflection angles shall be quoted in degrees to the nearest one-tenth of a second.

2.3 VERTICAL ALIGNMENT

2.3.1 Vertical Curves

2.3.1.1 Wherever possible vertical curves shall be positioned to avoid the coincidence with horizontal curves and, in particular with horizontal transitions. Where such coincidence is unavoidable, the largest practicable vertical curve radius shall be employed and it shall begin and end beyond the limits of the transition.

2.3.1.2 Vertical curves shall for each location be selected on the basis of the most suitable radius of the following: -

Table 1	
Absolute Maximum	25000m (constructability)
Desirable Maximum	3000m
Desirable	2500m
Minimum (main line)	2000m
Minimum (non-main line)	1600m
Table 2	
Regular vertical radius	$R_v \geq 0.4 \times V^2$
Minimum vertical radius	$R_v \geq 0.25 \times V^2$

Where,

V = Operating Speed (km/h)

Minimum length of vertical curves = 20m

2.3.1.3 The length of the constant grade between consecutive reverse vertical curves shall be as follows: -

Desirable minimum 50 m

Absolute minimum 20 m

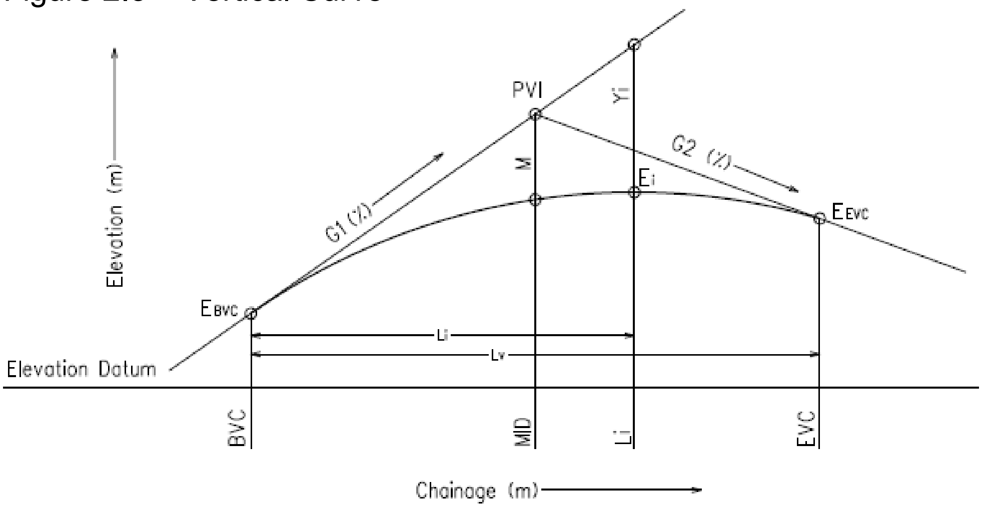
Consecutive vertical curves of same direction do not need a section of constant grade and may be connected directly.

2.3.1.4 Vertical curves shall not coincide with any part of the overall length of switches or crossings.

2.3.1.5 At station ends where vertical curves are provided in conjunction with acceleration/deceleration gradients, the tangent point of the vertical curve may be permitted only under severe constraints of the alignment to encroach within the length of the platform to a limited extent. This length of encroachment shall be such that the vertical offset of the curve from the station gradient at the platform end shall not exceed 15 mm.

2.3.1.6 Vertical curves shall be defined by parabolic curves having a constant rate of change in grade. The formula is as shown below:

Figure 2.3 – Vertical Curve



Where,

BVC	beginning of vertical curve radius (m)
EVC	ending of vertical curve radius (m)
PVI	point of vertical intersection
G1 / G2	rates of gradient, expressed in %
(G2 – G1)	the algebraic difference between the gradient (%)
L_v	horizontal length of vertical curve (m)
M	ordinate, offset from PVI to the curve (external distance) (m)
R_v	vertical curve radius (part of a parabola 2nd degree) (m)
Li	horizontal distance from BVC (m)
Yi	offset (m)
Ei	Grade elevation (m)

Equations:

$$\mathrm{L_v} = \frac{(\mathrm{G2} - \mathrm{G1}) \times \mathrm{R_v}}{100}$$

$$\mathrm{R_v} = \frac{\mathrm{L_v} \times 100}{\mathrm{G2} - \mathrm{G1}}$$

$$\mathrm{M} = \frac{\mathrm{L_v^2}}{8 \times \mathrm{R_v}}$$

$$\mathrm{Yi} = \frac{\mathrm{Li}^2}{2 \times \mathrm{R_v}}$$

$$\mathrm{E_i} = \mathrm{E_{BVC}} + \frac{\mathrm{Li} \times \mathrm{G1}}{100} + \mathrm{Yi}$$

2.3.1.7 Vertical curves can be omitted if the difference in adjacent gradients is less than or equal to 0.1%.

2.3.2 Gradients

2.3.2.1 The limit for gradients shall be as follows:

	Main line	Reception Track	Depot
Absolute Maximum	3.0%	3.0%	0%

The power rating of the traction motors and the operating speed shall govern the average maximum gradient distance longer than 500m.

2.3.2.2 At stations, the track shall be level throughout the platform length except for the limited lengths of vertical curves as specified in Clause 2.3.1.5 above.

2.3.2.3 A drainage gradient shall be provided for all underground tracks, other than at platforms and sidings, as follows: -

Desirable minimum 0.5%

Absolute minimum 0.25%

2.3.2.4 On ballasted track, level tracks may be employed provided drainage is catered for below the ballast.

2.3.2.5 Siding tracks should either slope 0.25% towards the buffers, or be level.

2.3.2.6 Where practicable within the bored sections of tunnels, acceleration/deceleration gradients shall be provided in the form of a hump profile between stations. The desirable value of the hump shall be 6m. Where tunnels are constructed by cut-and-cover methods, hump profiles need not be employed.

2.3.3 LEVELS

2.3.3.1 Levels shall be quoted in metres to three decimal places.

2.3.3.2 Rail level on canted ballasted track and slab track at grade & on above ground structure refers to the level at the crown of the lower inner rail (grade rail). Special attention is to be paid to abutting transition curves, where the connection of inner rails serves as reference.

2.3.3.3 Rail level on canted concrete slab track in tunnel refers to the midpoint between the two running rails and is unaffected by the application of cant.

2.4 TURNOUTS AND CROSSOVERS

2.4.1 Turnouts

2.4.1.1 Turnout design shall generally be based on the following standardised tangential geometry: -

Switches with continuous curve (curved common crossing)	Switches with straight end part (straight common crossing)	Maximum Permissible Speed (km/h)
60E1--140-1:6	60E1--140-1:7	34
60E1--190-1:7	60E1--190-1:9	40
60E1 300-1:9	-	50
60E1--500-1:12	60E1--500-1:14	65
60E1--760-1:14	-	80
60E1-1200-1:18.5	-	100

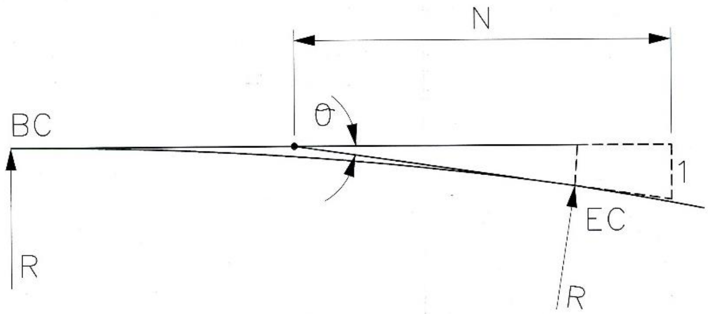
The turnout type shall be selected to suit the operation speed and headway requirement.

The turnout maximum permissible speed is based on 100mm cant deficiency or $a_q = 0.65$ m/s^2

Turnout radius 140m is not applicable to the existing North South Line, East West Line, North East Line, ballasted track and reception track.

2.4.1.2 The crossing angles of the turnouts are determined according to the Right Angle Measurement (RAM) as shown below:

Figure 2.4 – Right Angle Measurement (RAM)



$$\theta = \tan^{-1}\left[\frac{1}{N}\right]$$

$$N = \frac{1}{\tan\theta}$$

Where,

θ = angle in degrees

BC = begin of curve

EC = end of curve

R = Radius of the turnout measured in the centreline of the track

2.4.1.3 All turnouts shall not coincide with horizontal transition and/or vertical curves, and shall be avoided in circular curves wherever possible. Turnout shall not be placed across tunnel sections with differential settlements. Turnout shall not be placed at movement joints and designer shall engage track/structure designer for the minimum distance from the movement joint.

2.4.1.4 Drawings should state co-ordinates of the intersection point (IP) of turnouts, the chainage of beginning (BC), fouling point (FP) and both ends of turnout.

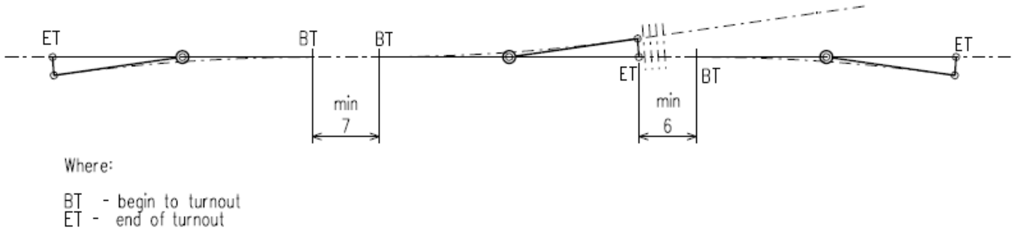
2.4.1.5 Turnout is a source of noise & vibration and its location shall be selected to minimise environmental impact.

2.4.2 Distance to Adjacent Turnouts

2.4.2.1 Distance between adjacent turnouts shall be designed to consider factors such as third rail electrical gapping, signalling, train stabling, insulated rail joint, future maintenance issues and track stability.

2.4.2.2 The minimum distance between the begin of turnout and the end of adjacent turnout shall be 6m. If the begin of turnouts are located facing each other the distance shall be at least 7m.

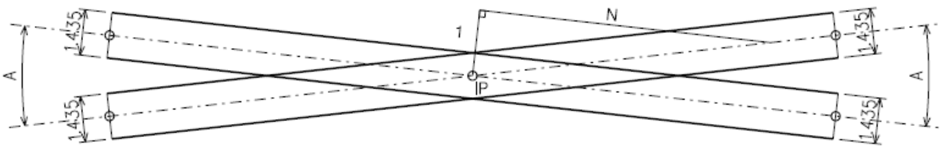
Figure 2.5 – Minimum Distance to Adjacent Turnouts



2.4.3 Diamond Crossings

2.4.3.1 Diamond crossings shall not be located in main lines except at terminal stations and depots. A number of standardised diamond crossings are listed in the table below.

Figure 2.6 – Diamond Crossing



Diamond Crossing	
Inclination 1 : n	Angle of inclination Degree (º ‘ “)
1 : 2.9167 (2 x 1:6)	18º 55’ 28.7”
1 : 3.4286 (2 x 1:7)	16º 15’ 36.7”
1 : 4.4444 (2 x 1:9)	12º 40’ 49.4”
1 : 5.9583 (2 x 1:12)	9º 31’ 38.2”

2.5 STRUCTURE GAUGE AND CLEARANCES

2.5.1 Definitions

2.5.1.1 The *normal co-ordinated axes of a vehicle* are defined as those orthogonal axes, normal to the longitudinal centreline of the vehicle, where one axis called the *wheel line* is the line connecting the points of bearing of pairs of wheels on the rails and the second, perpendicular to the first, called the *vehicle centreline*, is central between the wheels.

2.5.1.2 The *normal co-ordinated axes of the track* are defined as those orthogonal axes, normal to the longitudinal centreline of the track, where one axis, called the *rail line* is the common tangent to the tops of the rails and the second perpendicular to the first, called the *track centreline*, is central between the rails.

2.5.1.3 The *static load gauge* (also known as the Static Vehicle Profile) is defined as the profile related to the theoretical normal co-ordinated axes of the passenger vehicle outside which no part of the vehicle shall protrude when the vehicle is stationary and unloaded and when all play in the axles and suspension are uniformly distributed either side. Building tolerances for the vehicle are included in the static load gauge.

2.5.1.4 Horizontal *throw* is the distance measured parallel to the rail line of the vehicle centreline from the track centreline when a vehicle is on a horizontal curved track.

Horizontal throw reaches (arithmetic) maximum midway between bogies and at the ends of the vehicle. These throws are called *centre throw* and *end throw* respectively.

2.5.1.5 *Vertical* throw is defined in a similar manner when a vehicle is on vertically curved track. The maximum vertical throw at the minimum vertical radius shall be accounted for within the structure gauge.

2.5.1.6 The *Kinematic Load Gauge* (also known as the Dynamic Vehicle Profile) is defined as the vehicle profile related to the designed normal co-ordinated axes of the vehicle which covers the maximum possible distances from the vehicle centreline to any part of the vehicle taking into account the most unfavourable positions for running, including tolerances and wear.

2.5.1.7 The *Kinematic Envelope* is defined as the profile related to the designed normal co-ordinated axes of the track which covers the maximum possible distances from the track of any part of the vehicle taking into account the most unfavourable positions for running, including tolerances and wear of the track. When enlarged horizontally and vertically on curved track to allow for throw, it is referred to as the *Swept Envelope*.

2.5.1.8 The *Structure Gauge* is defined as the profile related to the designed normal co-ordinated axes of the track into which no part of any structure or fixed equipment may penetrate, taking into account all deformations and movements.

2.5.1.9 The *Service Vehicle Load Gauge* is the Kinematic Load Gauge for those rail vehicles used for construction and maintenance

2.5.1.10 The *Construction Gauge* is the structure gauge, which shall apply during construction until the time that trial running commences.

2.5.2 Train and Track Vehicles

All rail vehicles used for construction and maintenance shall conform to the service vehicle load gauge.

2.5.3 Structure Gauge

2.5.3.1 The Structure Gauge shall be based upon the Kinematic Envelope in such a way that each point on the perimeter of the Kinematic Envelope is enlarged vertically upwards by 70mm and horizontally by 100mm (two points to be constructed for each point on the Kinematic Envelope). Below the vehicle, the Kinematic Envelope is enlarged by 20mm to form the lower limit of the Structure Gauge. The Structure Gauge is the largest envelope based on the points constructed as described above. The shortest distance between the Kinematic Envelope and the Structure Gauge at any point is the *Clearance* at that point.

2.5.3.2 Special provisions must be made to permit the intrusion of the platform nosing and the platform screen doors into the Structure Gauge.

2.5.3.3 The Structure Gauge for curved track shall in all cases include an allowance for the maximum vehicle throw, both horizontal and vertical at the location being considered in accordance with Clause 2.5.4.1.

2.5.4 Throw

2.5.4.1 Horizontal throw can take the form of either centre throw or end throw. They are inversely proportional to the curve radius. When a vehicle is fully on a circular curve, throw may be calculated from the following formulae.

The throw is a linear function of vehicle dimensions and radius and can be expressed as:

Centre Throw, T_c

$$CC = (A^2 + B^2) / 8$$

$$= (2.5^2 + 15.8^2) / 8 = 32.0$$

$$T_c = CC / R = 32.0 / R$$

End Throw, T_e

$$CE = (N \times (B + N) - A^2 / 4) / 2$$

$$= (4.0 \times (15.8 + 4.0) - 2.5^2 / 4) / 2 = 38.82$$

$$T_E = CE / R = 38.8 / R$$

Where,

A = Axle distance, 2.5 m

B = Distance between bogie centres in metres, 15.8 m

CC = Centre throw constant

CE = End throw constant

N = End Section (from pivot point), 4.0 m

R = Radius in metres

2.5.4.2 Throw on transition curves and on switch and crossing and the adjacent tracks shall be calculated where necessary.

2.5.4.3 For simplified approach, the following figures shall be applied: -

Figure 2.7 – For End throw

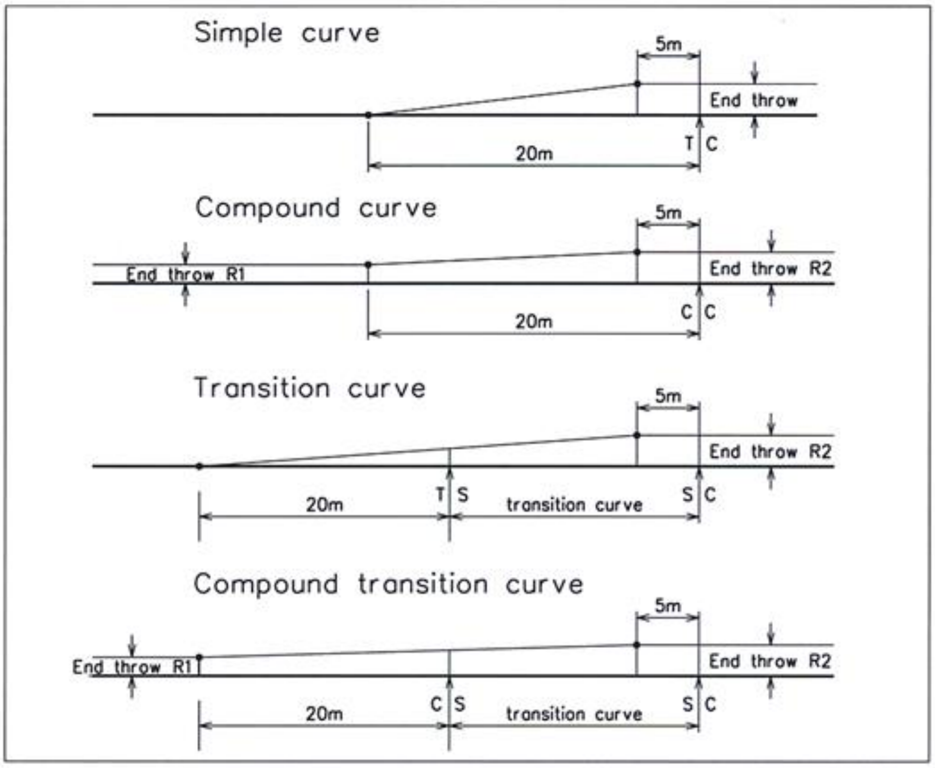
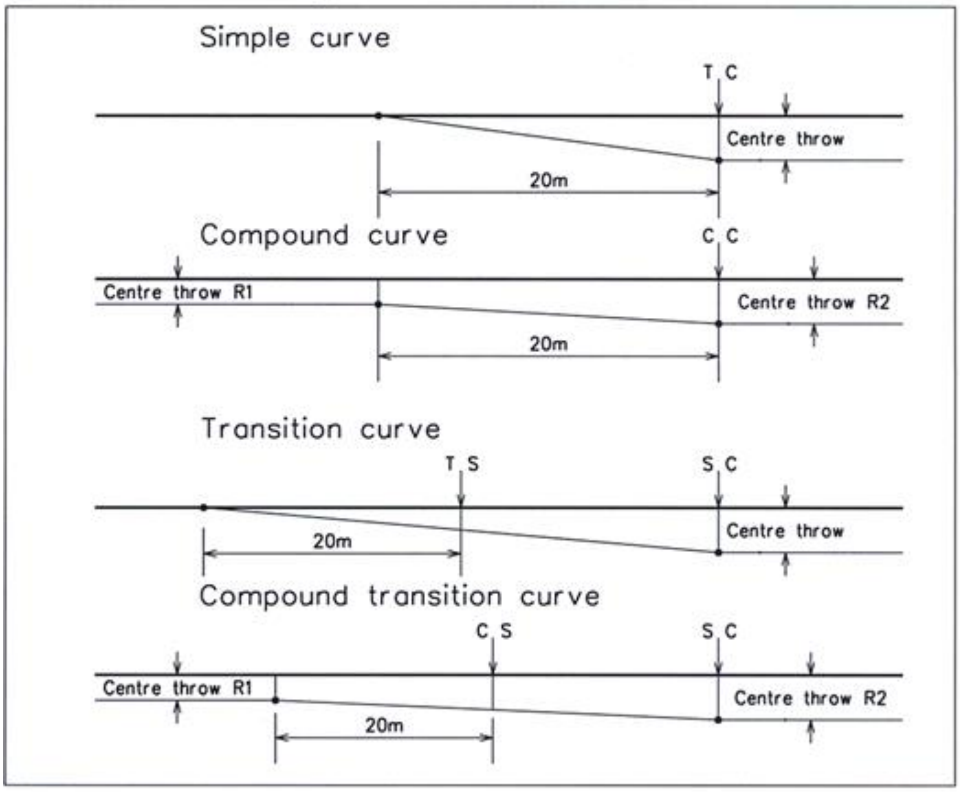


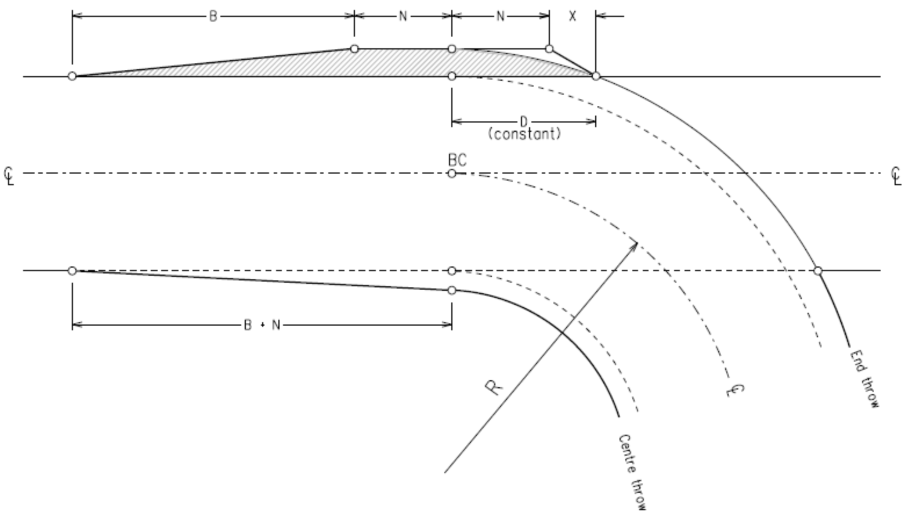
Figure 2.8 – For Centre throw



2.5.4.4 Throw at Turnouts

The throw at turnouts is to be widened as shown in the following: -

Figure 2.9 – Throw at Turnout



$\mathrm{D} = \sqrt{\mathrm{CE} \times 2}$ or $\sqrt{(\mathrm{R} + \mathrm{T_E})^2 - \mathrm{R}^2}$; whichever is greater

Where,

B = Bogie distance

D = Distance

N = Overhang

R = Radius of turnout

CE = Constant of end throw

T_E = CE / R

The distance D is constant for all turnout radii within the same MRT line, however the amount of end throw varies according to the turnout radius.

The calculation below illustrates an example of throw where;
B = 15.8m, N = 4.0m, R = 190m, CE = 38.8, T_E = 38.8/190 = 0.204m

$$\mathrm{D} \approx \sqrt{\mathrm{CE} \times 2} \approx \sqrt{38.8 \times 2} \approx \sqrt{(\mathrm{R} + \mathrm{T_E})^2 - \mathrm{R}^2} \approx \sqrt{(190 + 0.204)^2 - 190^2} \approx 8.8\mathrm{m}$$

2.5.5 Clearance to Structure Gauge

2.5.5.1 All structure and equipment shall be designed to be clear of the Structure Gauge with adequate allowance made to take into account all tolerances of construction and fixing, and for all deflections and displacements.

2.5.5.2 All moveable equipment, hinged doors, windows, etc. close to the track shall be positioned so that they are not within the Structure Gauge at every position of movement. All covers to sumps, pits, etc. within the track slab shall not infringe the Structure Gauge when in the open position.

2.5.5.3 Where two tracks are side-by-side close to each other, the minimum distance between the two tracks shall be such that the structure gauge will not encroach into each other.

2.5.5.4 The fouling point is the point of contact of the two respective structure gauges and the corresponding throws.

2.5.5.5 The fouling point for slab tracks in depot may be separated at the point of contact of the two respective Kinematic Envelopes and the corresponding throws.

2.5.6 Clearances at Station Platform Edge

2.5.6.1 Alongside the station platform limited intrusion into the Structure Gauge of the platform edge and screen doors is permitted. Intrusions into the Structure Gauge permitted shall not extend beyond the platform length.

2.5.6.2 The limit of platform screen door edge or finished edge of platform without platform screen door shall be set at 1675mm from the track centreline (such that 75mm clearance is provided horizontally between the static load gauge and the platform edge).

2.5.6.3 Where a curved and/or canted track is less than 20 m from the platform edge, the clear distance shall be increased to account for effect of cant and throw. The distance shall be calculated precisely, for the worst position of the train.

2.5.6.4 Passageway, handrailings and staircases beyond the platform ends shall be designed to be clear of Structure Gauge and taking effects of throw and cant into consideration where necessary.

2.5.6.5 In determining Kinematic Envelope and subsequently the Structure Gauge, the wear limit on the track system and the wheel shall be referred to the Code of Practice on the Maintenance of Permanent Way and Running Gear, Suspension System and Ride Quality in the Rolling Stock Engineering Standard. As specified in the Code of Practice on the Maintenance of Permanent Way, top wear of the rail shall not exceed 16mm. Side wear at the rail gauge corner of the rail shall not exceed 20mm measured to the vertical at 45°.

2.5.7 Clearances at Depot Facilities

2.5.7.1 Alongside depot platforms, intrusions into the Structure Gauge are permitted. Where the curved track is at least 20m beyond, the platform edge shall be set at minimum: -

a) 1715mm (+20 – 0 mm) from the track centreline for stabling tracks, rolling stock workshop, manual train wash, test track; or

b) 1660mm (+20 – 0 mm) from the track centreline for locomotive workshop and permanent way workshop. This requirement is based on the widest maintenance vehicle of 3100mm and passenger vehicles are not allowed to enter the track.

2.5.7.2 In addition, those features below platform level such as staircase, access ramp, services below platform, etc. shall be positioned beyond the minimum distance mentioned above with adequate allowance for all tolerances of construction and fixing, and for all deflections and displacements.

2.5.7.3 Where a curved and/or canted track is less than 20m from the platform edge, the clear distance with respect to the track centreline shall be increased to account for effect of cant and throw.

2.5.7.4 All other structures within the rolling stock workshop, locomotive workshop, permanent way workshop, manual wash plant building and features above platform level such as handrail, staircase, service pole, etc. shall be designed to be outside of the Kinematic Envelope with adequate allowance for all tolerances of construction and fixing, and for all deflections and displacements.

CHAPTER 3

ACTIONS

3.1 SCOPE

3.1.1 Actions shall be determined from SS EN 1991 and its Singapore National Annex (NA) except where stated otherwise in this Chapter. In any circumstances where there is a discrepancy between the relevant standards and regulations the more critical case shall apply.

3.1.2 The actions given are unfactored (nominal or characteristic) actions unless specifically noted otherwise.

3.1.3 For requirements of partial factors and actions specific to the type of structure, reference shall be made to the relevant Chapters.

3.1.4 For structures considered susceptible to aerodynamic effects (e.g. cable-stayed and suspension bridges), design criteria for wind actions shall be specially established to the Engineer's approval, and where necessary, the requirements shall be verified by prototype or model testing.

3.2 RTS ACTIONS

3.2.1 General

RTS Load Model shall comply with NA to SS EN 1991-2.

3.2.2 Derailment Actions

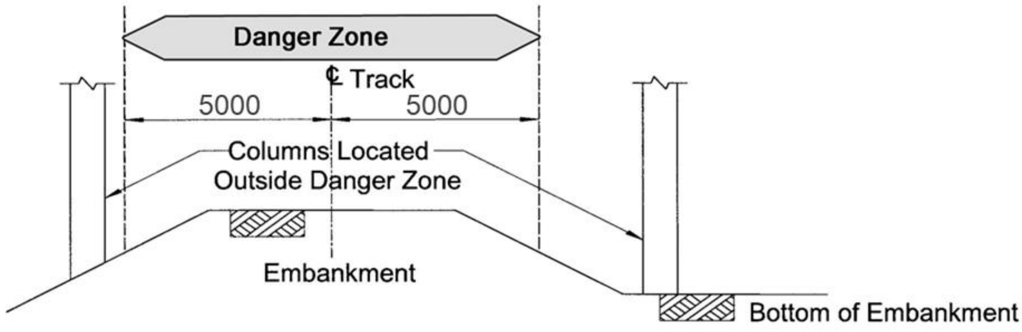
3.2.2.1 General

3.2.2.1.1 All structures subjected to impact from derailed railway traffic shall be classified as Class A structures. Supporting structures within danger zone (≤ 5.0m from centreline of track) shall be designed for impact force as specified in SS EN 1991-1-7 and its National Annex and UIC 777-2R. Danger zone definition shall be in accordance with UIC 777-2R. The design shall also comply with UIC 777-2R for progressive collapse.

3.2.2.1.2 Where supports must be placed inside the danger zone, they should be part of a monolithic structure (i.e. frame structural system).

3.2.2.1.3 Adequate protection measures shall be adopted to safeguard columns and piers located within or at the bottom of embankments, even if they are outside the danger zone because of the possibility of derailed vehicles rolling down the embankment. See Figure 3.1.

Figure 3.1 – Section View of "Danger Zone" within Embankment



3.2.2.1.4 Where individual columns are used within the danger zone a solid 'deflector' plinth shall be provided to a minimum height of the more onerous of the following:

a) 900mm above the rail level; or

b) 1200mm above ground level.

3.2.2.1.5 The height of the plinth shall be constant and the ends of the plinth shall be suitably shaped in plan to deflect derailed vehicles away from the columns.

3.2.2.1.6 For individual columns within station areas, a continuous wall supporting platform shall be used to provide similar protection.

3.2.2.2 Train Impact Forces

3.2.2.2.1 All design shall allow for the following minimum values of forces and design parameters due to derailment. More onerous values shall be incorporated where appropriate.

3.2.2.2.2 All piers, columns or walls, whose nearest face defines the boundary of, or lies within, the danger zone, shall be designed for train impact forces. These design values and their points of application shall be in accordance with UIC 777-2R and Group allocation shall be in accordance with Table 3.1. Annex A of National Annex to SS EN 1991-1-7 shall not be used

Table 3.1 – Substructure within Zone 2 Group Allocation

	Group as defined by UIC 777-2R
MRT	Group 1
MRT Depot	Group 3
MRT Test Track	Group 1

Impact force for LRT shall be 50% of impact force specified in UIC 777-2R

3.2.3 Imposed Loads in RTS Stations

3.2.3.1 Floor Loads

3.2.3.1.1 Floors within a station structure shall be under the following categories as defined in SS EN 1991-1.

Table 3.2 – RTS Station Floor Categories

Floor area usage	**Categories as defined in SS EN 1991-1**
For railway purposes (e.g. platform and concourse levels) and areas accessible to public during emergency	C5 (Areas susceptible to overcrowding)
For shopping and office purposes	B and D as appropriate

3.2.3.1.2 The minimum unfactored floor imposed loads shall be in accordance with SS EN 1991-1 except where otherwise specified below.

Table 3.3 – Floor Imposed Loads

Floor area usage	**Distributed Load (kN/m^2)**	**Concentrated Load* (kN)**
Areas susceptible to overcrowding	5	15
Traction and service substations, generator rooms	16	25
All other plant rooms	7.5	15

* Concentrated load shall be applied on a square area of 300mm side.

3.2.3.1.3 Where the actual extreme weights of equipment are known and verified by the relevant specialists and designers, they shall be adopted. The loads shall be taken as unfactored.

3.2.3.1.4 The maximum allowable equipment weight and co-existing distributed load shall be clearly indicated on the drawings. The loading arrangement which show the areas where the equipment load and co-existing distributed load are applied shall also be indicated.

3.2.3.1.5 Notwithstanding the requirements of SS EN 1991 and the above requirements, all floors shall be designed for the following loads: -

a) The total self-weight of a piece of equipment at any reasonable position likely to be experienced during or after installation. Access routes and method of transportation of the equipment during installation and any subsequent removal for repair shall also be considered.

b) The dynamic effect due to the operation of the equipment in its designed location.

3.2.3.2 Escalators

Approximate sizes and loads are given below. These sizes, number of intermediate supports and loads shall be verified by the specialist contractor and the design adjusted accordingly.

Table 3.4 – Escalator Approximate Size

Approximate size of section (mm)			**Approximate Weight (kg)**
Length	**Width**	**Height**	
6000	1700	2600	4500

Table 3.5 – Escalator Approximate Loads

Escalator Rise (mm)	**Reaction (kg)**			
	Lower landing	**Upper landing**	**Intermediate support 1**	**Intermediate support 2**
Above 11000	0.33H + 5400	0.37H + 6500	0.93H + 6900	0.93H + 6500
Up to 8000	0.49H + 4700	0.52H + 6500	1.73H + 7100	–
Up to 6000	1.67H + 6900	1.67H + 7500	–	–

Note: H is Rise in mm.

3.2.3.3 Handrailing and Balustrades

Imposed loads on handrailing and balustrades shall be as follows:

Floor area usage	**Horizontal UDL (kN/m)**	**UDL applied to infill (kN/m^2)**	**Point Load applied to part of infill (kN)**
Areas susceptible to overcrowding (public assembly areas *)	3.0	1.5	1.5
Areas accessible to maintenance staff only including those along edge of railway viaducts and railway platform end stairs	0.75	1.0	0.5

* includes areas accessible to the public during emergency.

3.2.4 Wind Actions

Approximate design actions and requirements for the design of fans and doors in underground RTS structures are given in the table below. These actions shall be verified by the relevant Specialist Contractor and the design adjusted accordingly.

Table 3.6 – Wind Actions for RTS Structures

Elements	Wind Actions and Requirements
Elevated RTS Structures including noise barrier	a) Wind forces on the structures shall be determined in accordance with SS EN 1991-1-4 and its Singapore National Annex. The fundamental value of the basic wind velocity shall be 25m/s (10-minutes mean wind speed).
Fans and related elements in Underground RTS Structures	a) Differential pressure of ±3.0kN/m^2 shall be allowed for in the design of the structures and elements, due to the operation of tunnel ventilation fans and the under-platform exhaust fans. It shall be applied to fan rooms, ventilation duct ways, plenums and shafts (including fitted doors and/or access hatches). b) Pressure drop not less than ±0.1kN/m^2 shall be allowed for across all louvre openings. c) Except for rooms used as fan rooms or air plenums, internal differential pressure of ±1.0kN/m^2 shall be allowed for between one room and the next, and ±0.3kN/m^2 shall be allowed between above and below close fitting false ceilings.
Doors in Underground RTS Structures	a) The door opening/closing mechanism for underground trainways including screen door areas shall be designed to operate under the condition of an overall differential pressure of ±1.0kN/m^2. b) Doors fitted to an air path used solely for draught relief, which leads from the atmosphere to a single running tunnel, shall be designed using a load of ±1.0kN/m^2 and a cycle load of ±0.5kN/m^2 for six million cycles. c) Differential pressure of ±2.0kN/m^2 and a cycle load of ±1.0kN/m^2 for six million cycles shall be assumed for cross-passage doors between adjacent running tunnels.
Ancillary Structures (e.g. covered linkway)	a) The fundamental value of the basic wind velocity shall be 22m/s (10-minutes mean wind speed).

3.3 HIGHWAY ACTIONS

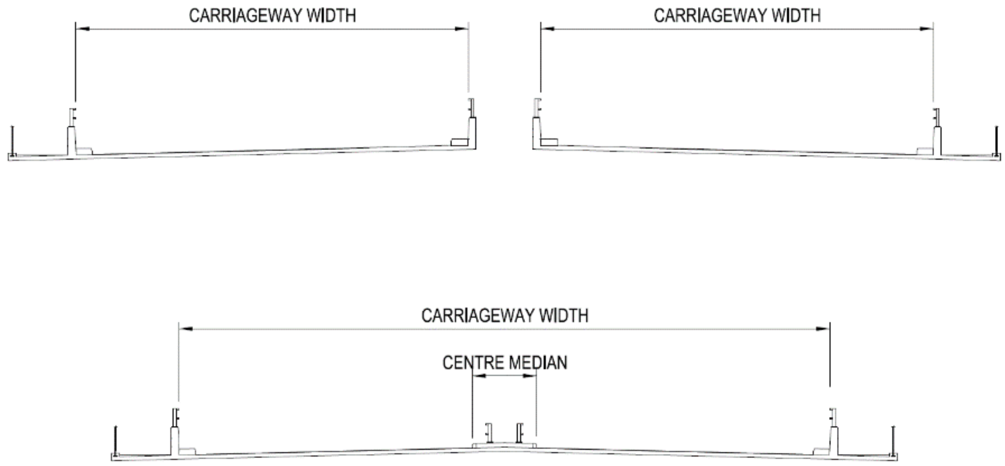
3.3.1 General

3.3.1.1 Highway actions shall comply with SS EN 1991 except where otherwise specified below:

3.3.1.2 Carriageway

The carriageway shall be that part of the running surface which includes all traffic lanes, hard shoulders, hard strips and marker strips. The carriageway width shall be the width between inner faces of outer parapets as shown in Figure 3.2. The carriageway width shall be measured in a direction at right angle to the line of parapets or raised kerbs. Carriageway shall include centre median/reserve if both the carriageways are on the same deck.

Figure 3.2 – Carriageway Width



3.3.2 Imposed Loads

3.3.2.1 Traffic actions shall comply with SS EN1991-2 and its Singapore National Annex. The design shall also comply with the following loadings:

a) Load Model 3 (LM3) and its application shall follow the NA to SS EN 1991-2. Only SV100 and SV196 need to be considered.

b) The SV100 and SV196 shall be used to determine the longitudinal braking and acceleration forces associated with LM3; and

c) Bus imposed load
   (i) 10kN/m^2 as unladen bus load for bus depots; and
   (ii) 15kN/m^2 as laden bus load for bus interchanges

d) Lateral loads on piers

Piers shall also be designed for a minimum horizontal force of 5% of the nominal vertical loads that consist of the permanent vertical load and ⅓ LM1 loading on one notional lane in each direction. This lateral load shall be multiplied by partial factors of 1.0 for the serviceability limit state and 1.4 for ultimate limit state and shall be applied in longitudinal and transverse directions as two separate load cases.

For the design of footing/foundation, the lateral load shall be applied at the footing/foundation level.

3.3.2.2 Traffic Actions for Underground Structures

Actions under roads shall be designed in accordance with PD6694-1 and the following loadings, or Clause 3.7.1 whichever more onerous: -

a) The top 200mm of cover shall be considered as road surfacing. γ_G for road surfacing shall be applied in accordance with SS EN 1990 and its Singapore National Annex.

b) An additional uniformly distributed load of 5.0kN/m^2 shall be applied. γ_G for superimposed dead load shall be applied in accordance with SS EN 1990 and its Singapore National Annex.

c) In the case of underground structures serving road vehicles (e.g. vehicular underpass and road tunnels), traffic loads inside and on top of the vehicular underpass shall be assumed to co-exist, with the exception that only one SV196 loading needs to be considered for any given load combination.

d) The SV196 shall be placed anywhere over and through the structure with co-existing LM1 loading on that carriageway, where more onerous. LM1 loading shall be simultaneously applied on all the other carriageways, where more onerous.

3.3.3 Wind Actions

Approximate actions for the preliminary design of internal walls, claddings and false ceilings in road tunnels are given in the table below. These actions shall be verified by the relevant Specialist Contractor and the design adjusted accordingly.

Table 3.7 – Wind Actions for Road Structures

Elements	Wind Actions and Requirements
Elevated road related structures including noise barrier	a) Wind forces on the structures shall be determined in accordance with SS EN 1991-1-4 and its Singapore National Annex. The fundamental value of the basic wind velocity shall be 25m/s (10-minutes mean wind speed).
Fans and related elements in Road Tunnels Facility Buildings	a) Differential pressure of ±3.0kN/m^2 shall be allowed for in the design of the structures and elements, due to the operation of tunnel ventilation fans. It shall be applied to fan rooms, ventilation duct ways, plenums and shafts (including fitted doors and/or access hatches). b) Pressure drop not less than ±0.1kN/m^2 shall be allowed for across all louvre openings. c) Except for rooms used as fan rooms or air plenums, internal differential pressure of ±1.0kN/m^2 shall be allowed for between one room and the next, and ±0.3kN/m^2 shall be allowed between above and below close fitting false ceilings.
Internal walls, claddings and false ceilings in Road Tunnels	a) Nominal pressures shall be ±1.5kN/m^2, applied in the combinations for wind actions as defined in Eurocodes. For fatigue on metal elements of cladding, the stress cycle values shall be taken as 50 million.
Ancillary structures (e.g. covered linkway)	a) The fundamental value of the basic wind velocity shall be 22m/s (10-minutes mean wind speed).

3.3.4 Collision Actions

3.3.4.1 The collision forces for substructures shall be determined using forces in Table 3.8 and applied with adjustment factors derived from the risk ranking procedure specified in NA to SS EN 1991-1-7. The annual average daily traffic (AADT) shall be taken as 106600 for expressway/semi-expressway and 10000 for all other carriageway.

Where risk ranking factors are not available in NA to SS EN1991-1-7, additional guidance may be referred to in PD 6688-1-7 for Pedestrian Overhead Bridge and BD51 for Gantry and High Level Covered Linkways.

Table 3.8 – Collision Forces on Substructures

Description	Component	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
Road Bridges, Railway Bridges, Pedestrian Overhead Bridges across Expressway (including lift shafts), Collision Protection Gantries	Main Component	1650	825	At the most severe point between 0.75m and 1.5m above carriageway level
	Residual Component	825	415	At the most severe point between 1m and 3m above carriageway level
Pedestrian Overhead Bridges across Other Roads (including lift shafts), Bus collision load within Bus Depots and Interchange	Main Component	* 250	* 250	At the most severe point between 0.75m and 1.5m above carriageway level
	Residual Component	* 165	* 165	At the most severe point between 1m and 3m above carriageway level

* Adjustment factors (F_a) not required

Note: Collision actions for Gantry and High Level Covered Linkways (HLCL) shall be in accordance with BD51.

3.3.4.2 Collision actions shall be considered even if vehicular impact guardrails are provided or even when there is no vehicular access to column positions.

3.3.4.3 Bridge piers situated in navigational channels shall be designed for ship collision forces. The forces shall be subject to approval by the relevant authorities. Protection system against such collision forces shall be considered.

Table 3.9 – Collision Forces on Superstructure within Road Reserve

Description	**Force F_{dx} in the direction of normal travel (kN)**	**Force F_{dy} perpendicular to the direction of normal travel (kN)**
Road and Railway Bridge < 5.7m clear height Pedestrian Overhead Bridge < 5.7m clear height (Expressway) Collision Protection Beams and Collision Protection Gantries	825	415
Minimum Robustness for Pedestrian Overhead Bridge Portal and Cantilever Gantries with Outreach Over Carriageway < 5.7m clear height High Level Covered Linkways < 5.7m clear height	85	85

3.3.5 Actions on Parapets and Railing

3.3.5.1 The design of parapet systems for bridges shall satisfy the requirement stipulated in this chapter and Chapter 9.

3.3.5.2 Vehicular parapet shall be designed for the ultimate forces specified in Table 3.10 and Table 3.11 or tested according to BS EN 1317 for Containment level N2 and H4a.

3.3.5.3 The forces for the design of pedestrian railings shall be in accordance with LTA Standard Details of Road Elements (SDRE).

Table 3.10 – Strength Criteria of N2 Parapet Metal Members and Supporting Post

Longitudinal Members	**Supporting Posts**
Product of plastic modulus and yield stress of 0.2% proof stress (kNm)	Transverse force F, to be applied (kN)
8.3 L/n	50

Notes:

(i) "L" equals the distance between centre line of supports in metres

(ii) "n" is the number of effective longitudinal members

Table 3.11 – Forces for The Design of N2 and H4a Full Height Concrete Vehicular Parapets

Item No.	**Description**	**Force**
Normal Containment (N2)		
1	Minimum ultimate moment of resistance, at the base of wall (see Note i) for bending in the vertical plane, with reinforcement adjacent to the traffic face (see Note ii)	25 kNm/m
2	Minimum ultimate moment of resistance for bending in the horizontal plane, with reinforcement adjacent to outer face (see Note ii)	12.5 kNm/m
3	Minimum ultimate moment of resistance of anchorage at the base of a pre-cast reinforced concrete panel	37.5 kNm/m
4	Minimum ultimate transverse shear resistance at vertical joints between pre-cast panels, or at vertical joints made between lengths of in situ parapet	65.5 kNm/m of joint

Table 3.11 – Forces for The Design of N2 and H4a Full Height Concrete Vehicular Parapets (Cont'd)

Item No.	Description	Force
Very High Containment (H4a)		
1	Minimum ultimate moment of resistance, at the base of wall (see Note i) for bending in the vertical plane, with reinforcement adjacent to the traffic face (see Note ii) a) For End sections (see Note iii) b) For intermediate sections (see Note iii)	 165 kNm/m 125 kNm/m
2	Minimum ultimate moment of resistance for bending in the horizontal plane, with reinforcement adjacent to outer face (see Note ii).	62.5 kNm/m
3	Minimum ultimate horizontal transverse shear resistance	220 kNm/m
4	Minimum ultimate transverse shear load to be transferred at vertical joints made between lengths of in situ parapet (see Note iv)	165 kN

Notes:

(i) The base of wall refers to horizontal sections directly above the bridge deck. The min. ultimate moments of resistance shall be reduced linearly from the base of wall value to zero at top of the parapet

(ii) In addition to the main reinforcement, in items 1 and 2 above, distribution steel equal to 50% of the main reinforcement shall be provided in the respective faces.

(iii) For design purposes, the parapet shall be divided into end sections extending a distance not more than 3m from ends of the parapet and intermediate sections extending along the remainder of the parapet.

(iv) The min. ultimate transverse shear load may be reduced to 50kN if the 3 longitudinal lengths adjacent and either side of the joint are designed as end sections. The shear load shall be uniformly distributed over the joint length

3.4 PEDESTRIAN AND CYCLIST LOADS

For structures serving pedestrians, the following loads shall be used:

Table 3.12 – Loads for Structures Serving Pedestrians

Types of Loadings		**Nominal loads (kN/m^2)**
Pedestrian and Cyclist Imposed Load		5.0
Imposed load on roof structures to be erected on structures serving pedestrians		0.5
Provision for future installation of roof structures	Dead Load	0.5
	Imposed Load	0.5

3.5 TRAFFIC ACTIONS ON TEMPORARY DECK STRUCTURES

The traffic actions on temporary deck structures shall be not less than the following: -

a) LM1 and LM2 loading as specified in SS EN 1991-2. Adjustment factors γ_{Qi} for LM1 shall be 1.0.

b) SV100 loading as specified in NA to SS EN1991-2.

c) Loading from construction vehicles. Any limits on construction vehicle loading shall be clearly indicated on the Temporary Works drawings.

The partial factors shall be applied in accordance with SS EN 1990 and SS EN 1991. There shall be no reduction in partial factors for loads or partial factors for materials.

3.6 THERMAL ACTIONS

Thermal actions shall be considered in accordance with SS EN 1990, SS EN 1991-1-5 and their Singapore National Annex except: -

a) Shade Air Temperature Range:

The shade air temperature range shall be taken as ±10°C from a mean temperature of 27°C. For these shade air temperatures, the extreme maximum and minimum mean temperature in concrete structures given in Table 3.13 shall be adopted.

b) Coefficient of Thermal Expansion:

The coefficient of thermal expansion for 1.0°C shall be taken as 12×10^{-6} for concrete.

c) Temperature Range for movement joints and bearings:

Bridge bearings and movement joints shall be designed for the temperature range given in Table 3.13.

Table 3.13 - Temperatures in Concrete Beam or Box Girders

Maximum and Minimum Mean Bridge Temperature (°C)	
Surfaced and Un-surfaced	
Maximum	Minimum
37	21

3.7 SURCHARGE LOADS

3.7.1 Surcharge loads shall be as given in Table 3.14. The surcharge load shall be applied at finished ground level (existing or proposed ground level, whichever is higher).

Table 3.14 – Minimum Surcharge Loads

	Minimum Surcharge Loads (kN/m^2)
Bored Tunnels	75
Temporary Works including earth retaining structures in temporary condition	20 #
Earth retaining structures in the permanent condition	25
All other structures	25

Note:

\# Loading from construction vehicles shall be considered if it is more onerous. Any limits on construction vehicle loading shall be clearly indicated on the Temporary Works drawings.

3.7.2 For structures influenced by loads from nearby building foundations or other structures, the following loads shall be considered: -

a) Self-weight of the existing structure with appropriate allowance for imposed load shall be applied as a surcharge at the foundation level of the building.

b) The surcharge loads shall be those for which the adjacent structure had been designed. In the absence of this information, the actual weights and imposed loads determined from the most onerous occupancy class for which the building is suitable shall be used. Where the effect of this load is less than the surcharge given in Clause 3.7.1, Clause 3.7.1 requirements shall govern the design.

3.7.3 Any known future works by others which may increase the loads on the structure shall be taken into account (e.g. earth filling in flood prone areas, reclamation works etc.) as agreed with the Engineer.

3.8 GROUND AND WATER LOADS

3.8.1 Soil Unit Weights and Earth Pressure

For the bulk densities of various types of soil, refer to Chapter 5. For the design parameters, appropriate horizontal coefficients and partial factors on actions to be used for respective Ultimate Limit State checks, refer to Chapters 5, 6, 7 and 8.

3.8.2 Ground Loads

The following additional loading requirements due to earth pressures shall be considered:

a) For assessing long term ground pressures, underground structures shall be considered as rigid structures subjected to “at rest” earth pressures.

b) In assessing ground pressures during construction, it shall be demonstrated that the pressures are compatible with the predicted ground movements.

c) Where appropriate, loads due to swelling (heave) of the ground shall be considered.

3.8.3 Design Ground Water Levels

3.8.3.1 Loads due to ground water pressure shall be calculated using a density of 10.00kN/m^3 and due to seawater using a density of 10.30kN/m^3.

3.8.3.2 The most critical of the following ground water conditions shall be considered for underground and earth retaining structures:

a) Design Ground Water Level (DGWL) shall be at the finished ground level or at a higher level depending on site conditions and the soil investigation results. Where there is seawater, the DGWL shall be the maximum high tide level of +2.35mSHD.

b) Water levels in Accidental Condition when there is a flood. The Design Flood Level shall be derived from the highest flood levels as agreed with PUB (Drainage).

c) For the design of bored tunnels, the Maximum Ground Water Load shall be determined from the Design Flood Level. The Minimum Ground Water Load shall be determined from the lowest credible groundwater level unless otherwise indicated.

3.9 ACTIONS FOR EQUIPMENT LIFTING FACILITIES

3.9.1 Crane Gantry Girder

Actions for crane gantry girder shall be in accordance with BS EN 13001.

3.9.2 Overhead Runway Beams

3.9.2.1 The working load of runway beams should be determined from the maximum weight of equipment to be lifted. Characteristic action shall be taken as 1.5 x working load, which includes an allowance for dynamic effects.

3.9.2.2 Fixings into concrete shall be designed to have an ultimate capacity of 3 x design load and giving consideration to the effects of non-axial load.

3.9.3 Eyebolts

3.9.3.1 Where eyebolts are used as fixed lifting points, they shall be designed in accordance with BS EN ISO 3266 subject to the following modifications to ensure yielding before brittle failure of the base material.

3.9.3.2 Loads

a) The safe working load of eyebolts for lifting or hauling shall be determined from the maximum weight of equipment to be moved. Eyebolts shall be selected/ designed in accordance with BS EN ISO 3266 giving consideration to the effects of non-axial load. Proof loading including dynamic effects shall be taken as 2 x the safe working load.

b) The maximum angle between the eyebolt axis and the line of application of pull shall be co-ordinated with the Specialist Contractors and clearly shown on the drawings.

3.9.3.3 Local anchorage

a) The anchorage capacity (e.g. pullout or cone failure) of the eyebolt fixing into the supporting member shall be designed for an ultimate load of 3 x safe working load.

b) In the case of concrete beams or slabs, the fixing shall be effectively anchored to the top of the supporting members using reinforcement links designed for this ultimate load.

3.9.3.4 Supporting Member

Supporting structural elements (for example slabs, walls, beams, etc.) shall be designed for a service load equal to the test load and for an ultimate design load equal to the test load multiplied by a partial safety factor for load equal to at least 1.4. The test load shall be taken as not less than 1.5 x safe working load.

CHAPTER 4

TRACKWORK

4.1 SCOPE

4.1.1 This chapter provides the minimum design requirements for trackwork and its interfaces with other elements. The interfacing elements include trackway, stations, structures, traction power, communications, signalling, rolling stock and drainage.

4.1.2 This design criteria is applicable only to steel wheel on steel rail mass Rapid Transit System (RTS).

4.1.3 In the design of the trackwork and selection of components, considerations shall be given to, but not limited to the following factors: -

a) durability;
b) reliability,
c) riding quality;
d) ease of construction and maintenance;
e) availability of track materials and components;
f) successful use of similar components in other transit railways;
g) compatibility with the rolling stock, electrical and signalling systems;
h) noise and vibration propagation to adjacent properties;
i) track alignment; and
j) electrical insulation & stray current control.

4.1.4 The design of trackwork and selection of components shall also consider the following operating and environmental considerations: -

a) Line functions (e.g. siding, maintenance facility);
b) Traffic (passenger trains, maintenance vehicles, gross annual tonnages, etc.);
c) Rail vehicles (axle loads, wheel diameter, wheel profile, wheel conditions, etc.);
d) Ambient temperature ranges;
e) Track Signalling Circuit requirements;
f) Traction Power Supply System requirements;
g) Potentially corrosive situations (e.g. wet tunnels, salty atmospheres, etc.).

4.2 TRACK

4.2.1 General

4.2.1.1 Track refers to the assembly of rails, sleepers, rail fastenings, and ballast or other forms of support, acting together to provide guidance and support for running of rail vehicles.

The design shall ensure stability, and line and level of the track under all load conditions.

4.2.1.2 Track gauge shall be 1435mm, measured between the running faces 14mm below the top of rails.

4.2.1.3 The design of the track shall take into account the provision allocated for other uses such as cable trough and pits, services, drainage sumps and walkway.

4.2.1.4 The track shall be designed to include loadings from: -

a) Permanent loads due to self-weight;
b) Vehicle axle loads;
c) Impact factors;
d) Lateral loads (wind, curving/hunting, nosing and centrifugal forces);
e) Longitudinal loads (braking, acceleration and thermal);
f) Derailment;
g) Environmental factors (e.g. thermal); and
h) Track-structure interaction.

4.2.1.5 Design Parameters

The track design shall as a minimum, include the following design parameters: -

a)	Axle Load (static):	160 kN
b)	Dynamic factor for all vertical loading:	1.5
c)	Minimum calculated safety factor:	1.5
d)	Maximum operating speed:	100 km/h

e) Maximum percentage of the load taken by a single bolt in a rail fastener with multiple anchors/bolts shall be: -
(i) 2 bolt system – 60%;
(ii) 3 bolt system – 50%; and
(iii) 4 bolt system – 40%.

f) Temperature range of rail outside tunnel shall vary from a minimum of 15°C to a maximum of 65°C;

g) Cable trough loading: -

(i)	Live load at mid-span (cover)	1.5 kN
(ii)	Lateral load on sides	6.0 kN/m

h) The longitudinal loading from braking and acceleration shall be: -

(i) Braking (kN/m) – 25% of consist maximum mass divided by the consist length; and

(ii) Acceleration (kN/m) – 20% of the consist maximum mass divided by the consist length.

4.2.2 Trackbed

4.2.2.1 The trackbed supporting the track may either be of ballast or ballastless (concrete slab). Concrete slab track shall be used in tunnels. Concrete slab track is preferred on elevated and at-grade tracks, unless otherwise specified by the Engineer.

4.2.2.2 **Concrete Slab Track**

4.2.2.2.1 2nd stage concrete refers to the second pour of concrete of minimum compressive strength class C30/37 to form the trackbed over the base concrete. The construction of the track shall be of a top-down method.

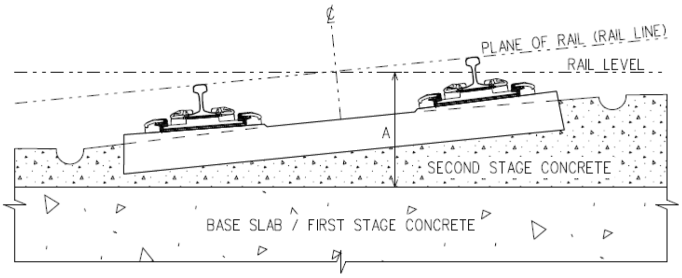
4.2.2.2.2 Where concrete slab track is provided, the interface between the trackbed concrete and the civil works shall be at different levels according to its location, as follows and Figure 4.1: -

a) For plain line tracks in main line, reception track and test track, the interface shall be 630mm below rail level;

b) For turnouts and crossover tracks in the main line where conduits for services pass under the track within the trackbed concrete, the interface shall be 850mm below rail level;

c) For tracks within the depot, the interface shall be 530mm below rail level

Figure 4.1 – Typical 2nd Stage Concrete



Location of Track	A (mm)
Mainline, Reception Track, Test Track	630
Crossover Tracks including Turnouts in Mainline	850
Depot Tracks	530

4.2.2.2.3 The above dimensions may vary nominally by ±20mm at the civil-trackwork interface

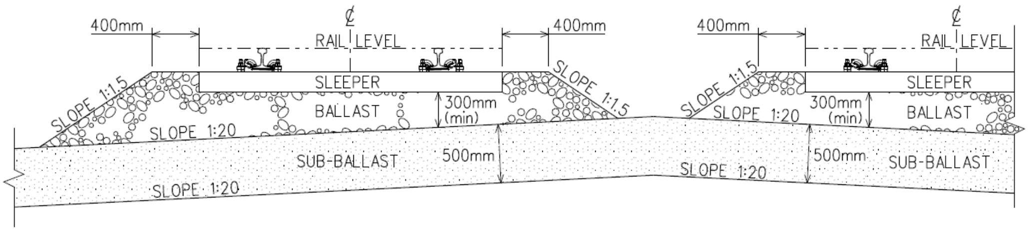
4.2.2.2.4 For track exposed to weather, the concrete slab track shall be designed to allow quick discharge of rainfall intensity for minimum 10-year return period, from the track and prevent water from flooding the rail fasteners.

4.2.2.2.5 Provision for stray current collector in 2nd stage concrete in the main line shall be in accordance with the requirements in Chapter 14.

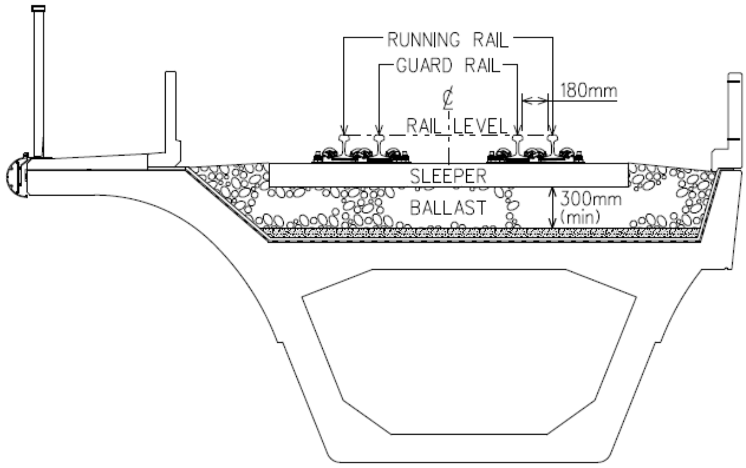
4.2.2.3 **Ballasted Track**

Sub-ballast and ballast shall provide stable support and drainage to the track. Crossfalls of 1:20 as shown in Figure 4.2(a) shall be provided in at-grade section. On weak or unstable ground measures shall be required to ensure stability of the sub-ballast prior to the laying of ballast and track.

Figure 4.2 – Ballasted Tracks



a) At Grade Cross Section



b) Viaduct Cross Section

4.2.2.3.1 Sub-ballast layer of 500mm thick shall be provided at track sections at grade, on embankments and in cuttings for main line as shown in Figure 4.2(a). The ballast thickness shall be at least 300mm thick measured between the bottom surface of the sleeper and the sub-ballast surface at the location of the lower rail, as shown in Figure 4.2(a) and (b).

4.2.2.4 **Track-Structure Interaction**

Track-structure interaction analysis, in accordance with SS EN 1990 and SS EN 1991-2, shall be carried out for tracks supported on bridge/viaduct to verify the compatibility of bridge/viaduct and trackwork designs. The track design shall consider: -

a) Permissible additional rail stress for ballastless track shall be 92MPa for both tension and compression;

b) Rail expansion joint shall be avoided;

c) Rail gap from a rail break shall be limited to 50mm;

d) Structural expansion joint of bridge/viaduct shall be minimum 10m away from the ends of turnout without affecting operation of switch point machine; and

e) Take into account structure temperature drop, residual creep and shrinkage.

4.2.3 Track Components

4.2.3.1 All components in the track shall conform to Eurocode, unless otherwise agreed by the Engineer and have a proven successful use for a period of at least five years in other transit railways.

4.2.3.2 Rails

4.2.3.2.1 Running rails shall be the 60E1 profile in accordance with BS EN 13674-1.

4.2.3.2.2 The application of running rails and the steel grades in accordance with BS EN 13674-1 shall be:

a) Rail steel grade R260:
   (i) straight track;
   (ii) curve track centreline of horizontal curve radius ≥700m; and
   (iii) all depot tracks.

b) Rail grade R350HT:
   (i) Curve track centreline of horizontal curve radius <700m and its connecting transition curves in main line; and
   (ii) Connecting tracks between heavily used turnouts at terminal stations and diamond crossings in main line

4.2.3.2.3 All running rails of plain line in main line and depot tracks shall be installed continuously welded and inclined at 1:40 inwards towards the track centre.

4.2.3.3 Rail Fastening System

4.2.3.3.1 Rail fastening system shall provide the support, guidance, stability, resilience and track-to-earth insulation to the running rails of the track, turnouts and crossings. It shall maintain a high and constant degree of gauge and rail inclination accuracies during operation.

4.2.3.3.2 Rail fastening system shall be categorised as ‘Category B’ fastening system in accordance with BS EN 13481-1.

4.2.3.3.3 Only indirect rail fastening system in accordance with BS EN 13481 shall be used in main line and depot tracks.

4.2.3.3.4 The rail fastening system shall incorporate a self-tensioning type of steel clip system that can hold the rail to the correct rail inclination, and fix the rail longitudinally by exerting a substantial and controlled toe load on the rail foot.

4.2.3.3.5 The baseplate shall be fixed to the sleeper/bearer by either:

a) two rail clips and cast-in anchors; or

b) cast-in bolts.

4.2.3.3.6 The rail fastening system in the main line and depot tracks shall use the same rail clip in tracks and turnouts for all locations.

4.2.3.3.7 Rail fastening spacing for track shall be designed to suit track geometry, loading, track stability, resilience, rolling stock characteristics, etc. Rail fastener spacing for track measured on the track centre line shall nominally be: -

a) 700 mm for both straight and curve tracks in main line; and

b) For depot tracks:

(i) 700 mm for ballasted track;

(ii) No greater than 1000 mm for concrete slab track; and

(iii) No greater than 1400 mm for pedestal track

4.2.3.3.8 The rail fastening system shall be designed to provide vertical adjustment for maintenance. It shall be capable in providing an incremental vertical height adjustment to a total of 14mm (minimum) at each rail support.

4.2.3.3.9 The rail fastening system shall be designed to give the longest electrical current leakage path to earth and highest rail to trackbed/support structure insulation. The electrical resistance of the rail fastener shall be in accordance with BS EN 13481.

4.2.3.3.10 Unless otherwise specified to meet noise and vibration requirements, the nominal static stiffness of rail fastening system in the main line shall be 21kN/mm, measured according to BS EN 13146-9.

4.2.3.3.11 The rail fastening system shall be designed to prevent retention of water or other deleterious matter when installed in any location.

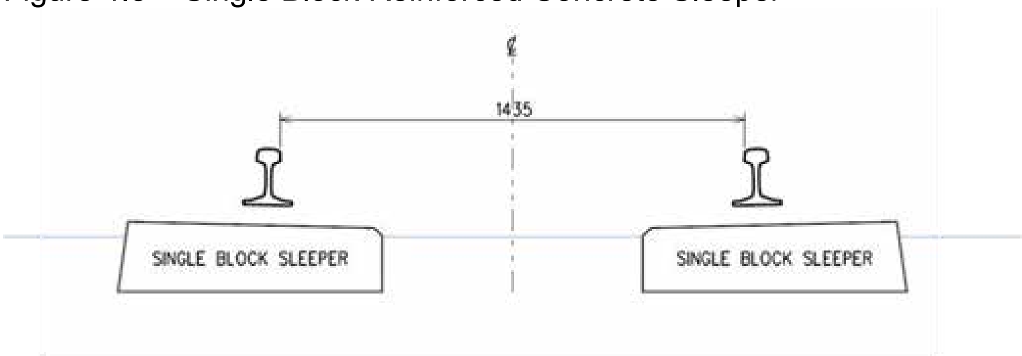
4.2.3.4 Concrete Sleepers and Bearers

4.2.3.4.1 Sleepers and bearers shall provide good anchoring of the rail fastening system, support to the rail and maintain a consistent track gauge. It shall be manufactured from concrete with minimum compressive strength class C45/55 in accordance to BS EN 13230 to provide durability and resistance to corrosion and ballast abrasion.

4.2.3.4.2 Prestressed monoblock concrete sleepers and bearers shall be used in all tracks. The design of prestressing strands shall consider the replacement of dowels/bolts/anchors of rail fastening system.

4.2.3.4.3 Single block reinforced concrete sleeper shall only be used in restricted areas such as drain sumps, subject to the approval of the Engineer.

Figure 4.3 – Single Block Reinforced Concrete Sleeper



4.2.3.5 Turnouts

Switches and crossing of turnouts shall be designed in accordance to BS EN 13232 to provide support and safe guidance of vehicle. It shall be designed to match the wheel diameters and profiles of rail vehicles, and switch machines controlled by the signalling system. All turnouts shall have the following characteristics: -

a) Turnout geometry shall be tangential in design;

b) The track gauge shall be 1435mm in all types of turnouts and crossings, measured 14mm below the top of rail;

c) Rail profiles and steel grades of various rails of the turnout shall be:

   (i) closure rail: 60E1 profile of grade R350HT in accordance with BS EN 13674-1;

   (ii) switch rail: 60E1A1 profile of grade R350HT in accordance with BS EN 13674-2;

   (iii) check rail: 33C1 profile of grade R260 in accordance with BS EN 13674-3

d) Switch rails shall have the following characteristics:

   (i) stress-free and fit snugly when in its closed position with the stock rails;

   (ii) manufactured from the asymmetric switch profile 60E1A1 forged to match the 60E1 rail profile. Welded joint within the entire length of switch rail is not allowed;

   (iii) opening of the switch toe shall not be less than 130mm;

   (iv) flangeway gap between the back edge of the opened switch rail and the running edge of the stock rail shall not be less than 55mm;

(v) sliding surface of the slide baseplate system shall be coated with low friction and lubrication-free material to transport the switch rail during operation. Roller shall be incorporated in the slide baseplate system in heavily used turnouts in the main line including the connecting reception tracks, and those located at terminal stations and turnback facilities; and

(vi) throw force to move the switch points with all operating and switch rods connected shall not exceed 2500N.

e) Common and obtuse crossings shall be of cast austenitic manganese steel type according to BS EN 15689, factory welded with R350HT grade rail steel weldable legs for joining to 60E1 rails by welding;

f) Rails in turnouts shall be vertical (no inclination); a gradual transition of rail inclination to match the connecting 60E1 rails shall be provided at each end of the turnout;

g) Rail fastener spacing for track measured on the track centre line shall not be greater than 700mm in turnout to suit the bearer layout;

h) Flangeway clearances through the turnouts and diamond crossings shall suit the wheel diameter, wheel profile, bogie arrangement and turnout/diamond crossing geometry;

i) All fabricated components shall be hot dip galvanised;

j) Devices to limit relative displacements between switch rails and stock rails or at crossings shall be provided;

k) Electrical resistance of fastening system shall be in accordance with BS EN 13481.

l) The track stiffness within turnout and diamond crossing shall be similar with that of the adjoining track. Where the difference in track stiffness between turnout and the adjoining track is greater that 3kN/mm, track stiffness of adjoining track shall be adjusted gradually to match the stiffness of the turnout; and

m) Drainage shall be provided to prevent standing water in depressed areas such as in flangeways, switch points and switch throwing mechanism.

4.2.3.6 Derailment Guard

Derailment Guard parallel to both running rails on elevated tracks shall be provided to keep derailed wheels adjacent to the running rails so as to prevent a derailed vehicle from striking fixed installations and parapets on viaducts. Guard rails shall be used in ballasted track and reinforced concrete upstands shall be used in concrete slab track.

a) Guard rails shall be: -
   (i) Of 60E1 of Grade R260 rails according to BS EN 13674-1, mounted on gauge side and not seated inclined;
   (ii) Provided in tracks adjacent to structural elements;
   (iii) Provided in all main line tracks on elevated structures;
   (iv) Fixed to double baseplate on alternate sleepers and curve inwards at the end of the line;
   (v) Terminated with entry and exit flares at turnouts and buffer stops;
   (vi) Electrically separated at 18m intervals by a gap of 50mm wide (unless otherwise required by signalling system) to avoid interference with the signalling system;
   (vii) Electrically insulated from the adjacent running rails; and
   (viii) Clear horizontal distance between the running rail and the adjacent guard rail shall be 180mm.

b) Reinforced concrete upstands shall be designed: -
   (i) to withstand a single horizontal point load of 100kN applied at the top of the upstand, perpendicular to the track centreline; and a vertical load of 25kN;
   (ii) with reinforcement bars of the concrete upstand that are within 500mm radius from top of rail, to be electrically insulated to avoid interference with the signalling system;
   (iii) to prevent impeding re-railing works.

4.2.3.7 Level Crossing

4.2.3.7.1 The rail fastening system used for level crossings shall be of an insulated type in accordance with clause 4.3.9. The gap between the concrete and the rail shall be sealed with pre-formed elastomeric flangeway sealing section.

4.2.3.7.2 Alternatively, the rail can be supported on a site-installed elastomeric compound, which provides resilience under the rail, lateral support and electrical insulation between the rail and the concrete channel.

4.2.3.7.3 All materials used shall be chemically resistant and electrically non-conductive, capable of taking vehicular load and suitable for use in the local environment.

4.2.3.7.4 Placing of switch/point machine within the level crossing is not allowed.

4.3 MISCELLANEOUS

4.3.1 Walkway

4.3.1.1 In main line tunnels, the space between the running rails is designated as the primary route for emergency evacuation. The track slab surface shall be non-slip and free from obstruction. Any unavoidable obstruction shall be suitably bridged with a non-slip material and ramped approaches.

4.3.1.2 In addition, a trackside walkway along the same side of the station platform shall be provided to permit evacuation through the side doors of trains. The trackside walkway shall take into account the floor height and stepping distance from the train in the tunnel.

4.3.1.3 The trackside walkway shall be of reinforced concrete, 380mm above the rail line and free of obstruction, at least 800mm wide and 2100mm headroom above the centreline of the walkway and have an even, non-slip surface. Ramps not steeper than a 1:12 gradient shall be provided in any change in level. Steps or ramp shall be provided to connect walkway to stations.

4.3.1.4 On the approach to a cross passage, ramp shall be provided to connect the trackside walkway and the cross passage to a level not greater than 210mm above the rail line. A reinforced concrete step unit, weighing not more than 50kg shall be demountable for maintenance of the drain between the walkway and rail.

A typical arrangement of the walkway at a cross passage is shown in Figure 4.4.

4.3.2 Buffer Stops

4.3.2.1 Sliding buffer stop, unless otherwise specified, shall be provided at the end of track to stop the train moving at speed of 18km/h on impact within the track occupancy distance.

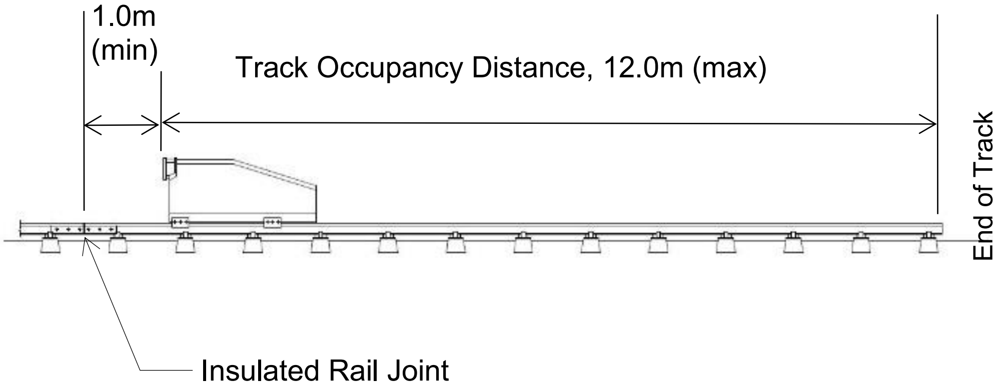
For trains with passengers, the deceleration rate shall be limited to 0.15g on impact. The deceleration rate on impact shall not exceed 0.22g for trains without passengers. Provisions for the sliding buffer stop shall be optimised to obtain the shortest possible track occupancy distance.

4.3.2.2 The buffer stop shall be designed to consider the track alignment geometry and coupler of train vehicle. The track over its track occupancy distance shall preferably be straight and on gradient not greater than 0.5%.

4.3.2.3 No weld is permitted in rails within the track occupancy distance.

4.3.2.4 For a 6-car train, the Track Occupancy Distance measured from the buffer head to end of sliding as shown in Figure 4.5, shall not be larger than 12m.

Figure 4.5 – Track Occupancy Distance



4.3.3 Cable Troughs

4.3.3.1 Cables shall be placed in cable brackets installed on walls of tunnels or viaducts, or routed through conduits embedded in trackbed.

4.3.3.2 Where cable trough is to be used, it shall be placed in between sleepers perpendicular to the track with its top surface flushed with the top surface of the adjacent sleepers.

4.3.3.3 For ballasted tracks sections on grade, cable troughs are to be installed parallel to the track with the cover in the same level as the lower edge of the sloped ballast surface.

4.3.3.4 Cable troughs used on ballasted tracks shall be designed to withstand weathering and shifting due to track maintenance and adjacent activities.

4.3.3.5 Cable trough shall be designed for water to be discharged freely and prevent stagnation of water within the cable trough.

4.3.4 Reference Points and Distance Indicators

Reference points shall be securely placed at permanent immovable locations alongside the track, unobstructed and easily readable. They shall contain the following data: -

a) Chainage value

b) Rail level

c) Alignment points (BC, EC, TS, SC, CS, ST, PVC, PVT), curve radius

d) Offset (Distance to nearest gauge face of running rail)

e) Cant value

f) Fouling point

g) Distance indicator

4.3.5 Cross Bonding and Rail Bonding Cables

4.3.5.1 For bridging insulated sections of the running rail in order to ensure the continuous flow of the negative return current, cross-bonding cables are required. In general, cross bonding cables between tracks and rail bonding cables at all turnouts in main lines shall be connected to the interrupted rails on either side of the insulated sections.

4.3.5.2 The installation locations shall be co-ordinated and determined in collaboration with the Signal and Power designers. The method of cable connection to the running rail shall be subject to the approval of the Engineer.

4.3.6 Bonded Insulated Rail Joint

4.3.6.1 All bonded Insulated Rail Joints (IRJ) shall be pre-fabricated under shelter condition and welded onto the rail. In-situ IRJ is only permitted where pre-fabricated joints cannot be welded in for geometrical reasons.

4.3.6.2 The IRJ is to separate electrically two adjacent running rail sections and shall have minimum electric resistance of 1000 ohm for a thoroughly moistened joint. The location shall be determined by the Signal and Power designers.

4.3.6.3 Bonded IRJ shall be of the glued synthetic-resin type with steel fishplates and fasten 60E1 rails with 6 numbers of high tensile lock bolts. The fishplate profile shall not interfere with train wheel throughout its service life. It shall have a proven record with a major railway system for a minimum of 5 years.

4.3.7 Rail Welding

4.3.7.1 Rail for all main line and depot tracks outside the limits of turnouts shall be welded with minimum 18m-long rail lengths into continuous strings using the electric flash-butt welding process in accordance with BS EN 14587 by means of an on-track machine.

4.3.7.2 Aluminothermic welding process in accordance with BS EN 14730 shall be used in locations within turnout, prefabricated IRJ, and in-situ welding of running and closure rails. Weld portion of higher grade shall be used to weld rails of dissimilar grades.

4.3.7.3 Excessive weld material shall be removed and ground to match the rail head on either sides of the weld.

4.3.7.4 All rail weld shall be indelibly marked to clearly identify the welder and the weld.

4.3.7.5 Rail weld shall not be located at structural movement joint, signalling null point and at the rail fastener.

4.3.8 Track Insulation

4.3.8.1 The running rails will be an integral part of the traction current return system and will be the means by which the traction current will be returned to the sub-stations. Accordingly, the rails shall be electrically insulated from earth as per BS EN 50122-2 to ensure the functioning of the signalling and the traction power supply systems.

4.3.8.2 Without any other trackside equipment or cable installation connected and under damp conditions, the track system shall meet the following requirements: -

a) A minimum of 10 ohm-km resistance between the track and sub-station water earth, with both running rails of the track electrically connected; and

b) A minimum running rail to running rail resistance of 2 ohm-km.

With trackside equipment and cable connected to the rails, under damp conditions, the track system shall meet the requirements in accordance with Chapter 14.

4.3.8.3 Any reinforcement used in the design of the trackbed, sleepers and bearers, should not form conductive loops up to 500mm radius from the top of rail. Insulators or non-conductive material shall be used to prevent any loop developing in the layout of the reinforcement within 500mm radius from top of rail.

4.4 THIRD RAIL SYSTEM

4.4.1 General

4.4.1.1 The third rail system shall be of the bottom contact type in which vehicle-mounted current collector shoes press upwards onto the underside of the conductor rail. The layout shall provide a continuous and uninterrupted traction power supply to the trains.

4.4.1.2 The third rail system shall take into account all service, dynamic, mechanical and electrical loads. Position of components, particularly the stainless steel contact surface shall be co-ordinated with the design of collector shoes.

4.4.1.3 The third rail shall be terminated at a minimum of 2m in front of any pedestrian track crossing or vehicular level crossing. Terminations of the third rail shall consider electrical sectionalisation in main line and at the various facilities, including underfloor wheel lathe and train maintenance workshop.

4.4.1.4 The complete third rail system, including the protective cover, shall be capable of withstanding a downward vertical load of 1.5kN and a shock load of 50kg steel ball free-falling 300mm, at any point without causing any permanent deflection when the load is removed.

4.4.1.5 All fastenings securing the conductor rail and insulator at the support shall be of stainless steel minimum grade A4-80. Forging of stainless steel components is not allowed.

4.4.2 Conductor Rail

4.4.2.1 The conductor rail shall be well supported to prevent lateral or vertical motion while allowing longitudinal expansion and contraction under varying thermal conditions.

4.4.2.2 The conductor rail shall be manufactured from a high-conductivity aluminium alloy with a stainless steel wearing face for contact with the train current collector shoes. The conductor rail shall have sufficient cross-sectional area to carry the required traction current. The stainless steel shall be joined to the aluminium either by a molecular or welding process, and not only by mechanical means.

4.4.2.3 The conductor rail shall be supplied straight and in standard 15m-long lengths. For curves with centreline radius of less than or equal to 150m, the conductor rail shall be supplied pre-curved in the factory.

4.4.3 Joints in Conductor Rail

Individual lengths of conductor rail shall be rigidly connected to each other, both mechanically and electrically, using splice plates made from the same aluminium alloy as the rail. The splice bars shall be fixed to the conductor rails by no fewer than four vibration-resistant lock bolts.

4.4.4 Ramps

4.4.4.1 Entry and exit ramps shall be provided at turnouts and at other locations where a gap in the conductor rail is necessary. They shall also be provided at all electrical disconnecting points and changes of the third rail installation from one side of the track to the other.

4.4.4.2 The ramp design shall take into account the differing speed requirements of a running train and minimise arcing. High speed ramp shall be used for main line tracks, reception tracks, test tracks and other locations where train speeds exceed 20km/h. Low speed ramps shall be used for train speeds is 20km/h or less.

4.4.4.3 Side entry ramp is not allowed.

4.4.5 Cable Terminals

4.4.5.1 Cable terminals shall be designed to carry safely the full rail current without overheating.

4.4.5.2 The installation locations of cable terminals shall be co-ordinated and determined in collaboration with the Signal and Power designers.

4.4.5.3 The materials of the cable terminals shall be selected to avoid electrical corrosion. Cable terminals shall be fixed to the conductor rails with vibration-resistant lock bolts.

4.4.6 Conductor Rail Supports

4.4.6.1 The conductor rail support shall be electrically insulated from the trackbed and designed to carry all static and dynamic loads in the system. It shall also withstand electromagnetic and thermal loads, and short circuits. Fasteners shall not loosen due to vibration from train operation.

4.4.6.2 Support spacing shall be designed to the lesser of: -

a) Limiting conductor rail to maximum vertical deflection of 1.5mm under all static loading condition; and

b) not greater than 5.6m

4.4.6.3 The claw assembly and the conductor rail support shall hold the conductor rail without twist, to maintain consistent conductor rail contact surface with the current collector shoes.

4.4.6.4 The conductor rail support shall be fixed onto the trackbed or sleeper. It shall allow vertical and horizontal adjustments of the conductor rail during train service without disturbing mountings on the support structure.

4.4.7 Expansion Joints

4.4.7.1 Expansion joints shall be provided to accommodate longitudinal expansion and contraction of the conductor rail. It shall be designed to avoid arcing, and incur minimal wear and maintenance. Intervals of expansion joints shall be designed in consideration of the environmental conditions.

4.4.7.2 Electrical continuity across the joint shall be maintained. Precautions shall be taken to ensure that the movement joint does not suffer from electro-chemical corrosion. The gaps of the expansion joint shall be bridged by an electrical shunt.

4.4.7.3 A mid-point anchor shall be installed mid-way between two consecutive expansion joints to prevent longitudinal sliding of the conductor rail.

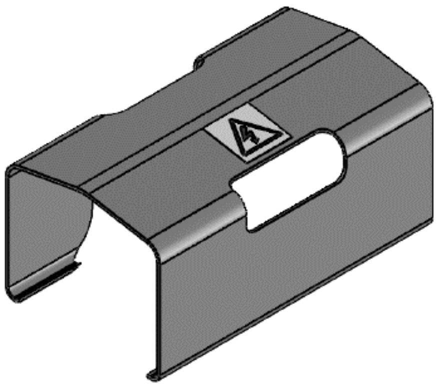
4.4.8 Protective Cover

4.4.8.1 The third rail shall be provided with a continuous insulating cover to protect persons from accidental contact with the third rail and protect the third rail from foreign objects falling or being thrown onto it.

4.4.8.2 Outside tunnels, the covers shall be resistant to degradation from ultraviolet radiation from the exposure to sunlight. The materials used for the protective cover shall be unplasticized polyvinyl chloride (UPVC) and glass fibre reinforced plastic (GF-RP) inside tunnels.

4.4.8.3 An opening in the insulator cover shall be provided to enable easy inspection of the support fastener without removing the protective cover.

Figure 4.6 – Insulator Cover with Opening



4.4.9 Insulator

4.4.9.1 The insulators shall be able to withstand all loads applied to the conductor rail and all likely over-voltages without damage or deterioration.

4.4.9.2 The insulators shall have a minimum resistance of 1GΩ at 500V D.C. under dry condition; and the surface finish shall have high surface tracking resistance.

4.4.9.3 In tunnel, insulators shall be manufactured from glass reinforced polyester.

4.4.9.4 For at-grade and elevated tracks, insulators shall be of cast cycloaliphatic epoxy resin, resistant to fatigue cracking and suitable for a tropical climate with exposure to ultraviolet rays.

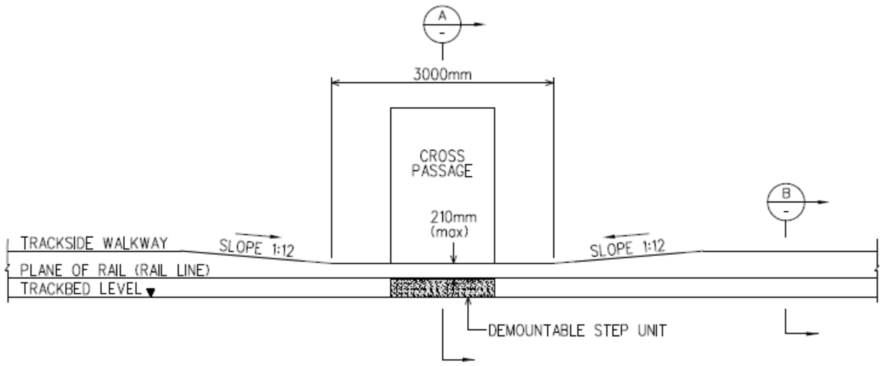
4.5 OTHER INTERFACING SYSTEMS

4.5.1 All information concerning the rolling stock to be used in the system shall be provided by the Authority and/or the rolling stock designer.

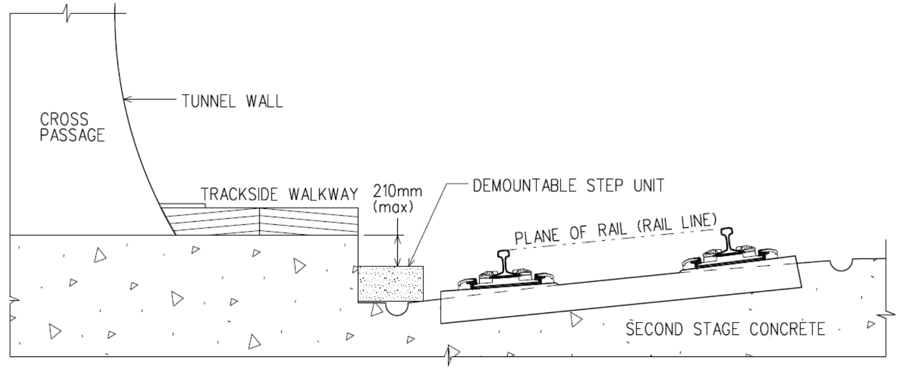
4.5.2 The signalling system, depending on the type of system to be supplied, may employ audio frequency track circuits that is connected to the running rails for train detection purpose.

4.5.3 Stray Current Control System (see Chapter 14)

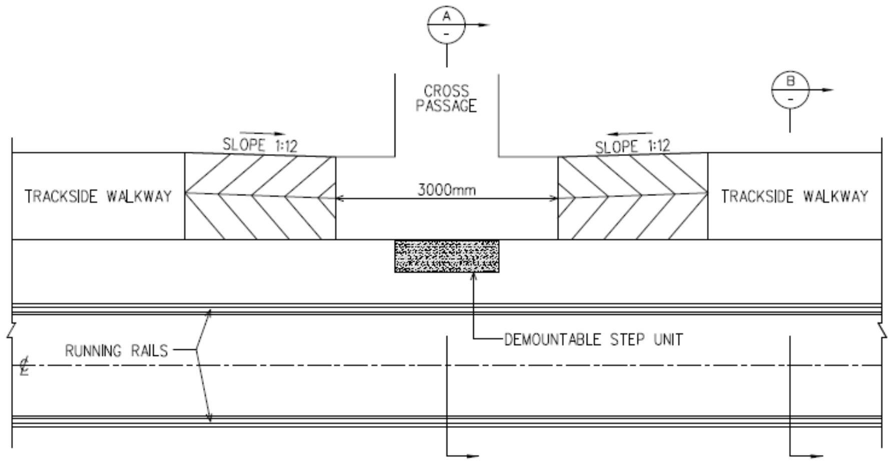
Figure 4.4 – Typical Walkway at Cross Passage



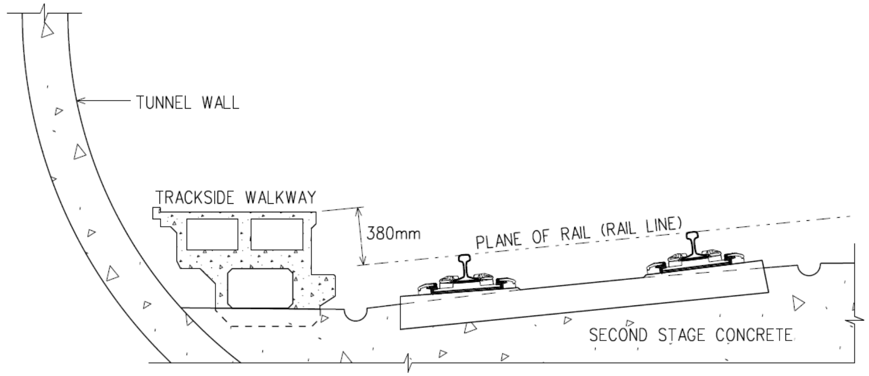
ELEVATION AT CROSS PASSAGE



SECTION A



PLAN VIEW AT CROSS PASSAGE



SECTION B

CHAPTER 5

GEOTECHNICAL PARAMETERS

5.1 GENERAL

The Geotechnical design parameters and other requirements/ information given in this chapter have been derived from various LTA projects.

5.2 HYDROGEOLOGY

5.2.1 Rainfall

Mean monthly rainfall values shall be based on the data from the Meteorological Service of Singapore. Please refer to the Meteorological Service of Singapore website for the updated Mean Monthly Rainfall values.

5.2.2 Design Ground Water Levels

Refer to Chapter 3.

5.3 SOIL AND ROCK CLASSIFICATION

The common soils and rocks of Singapore have been classified into a number of "Soil and Rock Types" relating to their geological origins as shown in Table 5.1 and Table 5.2.

Table 5.1 – Classification of Soil and Rock Types

REFERENCE	SOIL & ROCK TYPE	GENERAL DESCRIPTION	GEOLOGY OF SINGAPORE 2nd EDITION
B	BEACH (Littoral)	Sandy, sometimes silty, with gravels, coral and shells	KALLANG Littoral, possibly also part of all other members and TEKONG
E	ESTUARINE (Transitional)	Peats, peaty and organic clays, organic sands	KALLANG Transitional, possibly part of Alluvial and Marine
F	FLUVIAL (Alluvial)	Sands, silty or clayey sands, silts and clays	KALLANG Alluvial, possibly part of all other members and TEKONG
F1		Predominantly granular soils including sands, silty sands, clayey sands	Bed of Alluvial Member of KALLANG
F2		Cohesive soils including silts, clays, sandy silts and sandy clays	Bed of Alluvial Member of KALLANG
M	MARINE (Marine)	Very soft to soft blue or grey clay	Marine Member of KALLANG
O	OLD ALLUVIUM	Pebbles, coarse sand with fine pebbles, medium to coarse sand and clay and silt, variably cemented. See Table 5.3 for weathering classification	OLD ALLUVIUM

Table 5.1 – Classification of Soil and Rock Types (Cont'd)

REFERENCE	SOIL & ROCK TYPE	GENERAL DESCRIPTION	GEOLOGY OF SINGAPORE 2nd EDITION
FC	FORT CANNING BOULDER BED (previously known as S3, Bouldery Clay or Boulder Bed)	A colluvial deposit of boulders in a soil matrix. The matrix is typically a hard silt or clay, but can be granular. The material is largely derived from the rocks and weathered rocks of the Jurong Formation	Fort Canning Boulder Bed
S	SEDIMENTARIES (Rocks & associated soils)	Sandstones, siltstones mudstones, conglomerate and limestone. The rock has been subjected to a varying degree of metamorphism. See Table 5.3 for weathering classification	JURONG Tengah, St John, Rimau, Ayer Chawan, Jong, Queenstown and Pandan Facies
G	GRANITE (Rocks and associated Residual soils)	Granitic rocks, including granodiorite, adamellite and granite. See Table 5.3 for weathering classification	BUKIT TIMAH GRANITE

Table 5.2 – Rock Weathering Classification

GEOLOGICAL CLASSIFICATION			
MRTC (1983)	**LTA Guidance Note (2001)**	**GRADE/CLASS**	**DESIGNATION**
S1 & G1	GI & SI	I	Fresh
	SII & GII	II	Slightly Weathered
S2 & G2	SIII & GIII	III	Moderately Weathered
	SIV & GIV	IV	Highly Weathered
S4 & G4	SV & GV	V	Completely Weathered
	SVI & GVI	VI	Residual Soil
O	OA	A	Unweathered
	OB	B	Partially Weathered
	OC	C	Distinctly Weathered
	OD	D	Destructured
	OE	E	Residual Soil

Note 1: MRTC (1983) - a weathering classification system developed in 1983 for the first phase of the MRT. This system was based on the proposals made by the Geological Society Engineering Group Working Party in 1970 (Anon 1970), but simplified for use in Singapore. The use of this system shall be discontinued and replaced by LTA Guidance Note (2001).

Note 2: Weathering classifications based on BS 5930 (1999, 2015). See Appendix 5.1.

Note 3: Between 1983 and the issue of the guidance note, various systems for the classification of the weathering of the Old Alluvium were in use. Reference should be made to the corresponding reports for the basis of those classification systems.

Note 4: No weathering classification required for FC material.

5.4 DESIGN PARAMETERS

5.4.1 Minimum values for common design parameters for the soils and weathered rocks of Singapore are given in Table 5.3. The design shall be carried out using these parameters, or less conservative parameters provided the designer can demonstrate as being appropriate, to the acceptance of the Engineer, based on the results of the soil investigation for the Project.

5.4.2 The subgrade modulus for a given soil or rock depends on the length, width and the depth of the loaded area. These factors should be considered in establishing the design subgrade modulus for each type of foundation or retaining structure. Typically, the design modulus should be derived from the elastic modulus of the loaded soil/rock by establishing the relationship between the contact pressure and the resulting settlement or deflection, using an acceptable analytical method or numerical modelling. The modulus can also be obtained from plate load tests, with appropriate modifications to the scale of the test and depth effects.

5.4.3 The selection of other Design Parameters which are not given in Table 5.3 shall be derived from the site investigation carried out for the Project and from other relevant geotechnical exploration, sampling and testing. Design Parameters must be justified and submitted to the Engineer for acceptance.

5.4.4 The parameters given in Table 5.3 are for the design of permanent structures where the minimum strength is critical to the design. They are not intended for the selection of construction equipment where the maximum strength has to be considered.

Table 5.3 – Design Parameters

MRTC (1983) Classification		B	E	F1	F2	M	O					S3	Fill
Classification based on BS 5930 (1999) & LTA's Guidance Note		B	E	F1	F2	M	O Class A	O Class B	O Class C	O Class D	O Class E	FCBB	Fill
Design Parameters	Bulk Density (kN/m^3)	19	15	20	19	16	20	20	20	20	20	22	19
	Coefficient of Earth Pressure at Rest (K_0)	0.5	1.0	0.7	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	0.5
	Undrained Cohesion, C_U (kPa)	0	Figure 5.1 & Note 1	0	Figure 5.2 & Note 1	Figure 5.3 & Note 1	Note 2	Note 2	Note 2	Note 2	Note 2	Note 2	0
	Effective Cohesion, C' (kPa)	0	0	0	0	0	Note 3	Note 3	Note 3	Note 3	Note 3	10	0
	Effective Angle of Friction (degrees)	30	5	30	22	22	Note 3	Note 3	Note 3	Note 3	Note 3	28	30

Table 5.3 – Design Parameters (cont')

MRTC (1983) Classification		S1		S2		S4		G1		G2		G4	
Classification based on BS 5930 (1999) & LTA's Guidance Note		SI	SII	SIII	SIV	SV	SVI	GI	GII	GIII	GIV	GV	GVI
Design Parameters	Bulk Density (kN/m^3)	24	24	22	22	21	21	24	24	23	23	20	20
	Coefficient of Earth Pressure at Rest (K_0)	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8
	Undrained Cohesion, C_U (kPa)	N/A	N/A	N/A	N/A	Note 2	Note 2	N/A	N/A	N/A	N/A	Note 2	Note 2
	Effective Cohesion, C' (kPa)	Note 4	Note 4	Note 4	Note 4	0	0	Note 4	Note 4	Note 4	Note 4	0	0
	Effective Angle of Friction (degrees)	Note 4	Note 4	Note 4	Note 4	30	30	Note 4	Note 4	Note 4	Note 4	30	30

N/A: Not Applicable.

Note 1: Figures 5.1, 5.2 & 5.3 gives the undrained cohesion of the normally consolidated or slightly over-consolidated estuarine, fluvial and marine clay. For the areas under reclaimed land, the undrained cohesion of these clays shall be derived from in-situ tests, such as cone penetration tests or in-situ vane tests.

Note 2: Undrained conditions do not usually apply for deep excavations in these materials, but may be applicable during tunnelling. The methods outlined in Clough and Schmidt (1981) may be used to assess if undrained parameters are applicable. The design should be carried out for both drained and undrained parameters, and the more conservative of these designs should be adopted. For undrained analysis, a value of 5 x N (SPT value in blows/300mm) kPa, up to N = 50, may be adopted for the undrained cohesion in these materials, where applicable.

Note 3: Effective Stress parameters for the Old Alluvium shall be established for each site based on p'- q plots.

Note 4: Effective Stress parameters for these materials should be derived from site-specific data. The Geological Strength Index method (Hoek and Brown, 1997) is considered appropriate for this.

Note 5: The design of temporary ERSS can adopt a lower coefficient of lateral earth pressure at rest, K_0 = 0.7 for Old Alluvium materials.

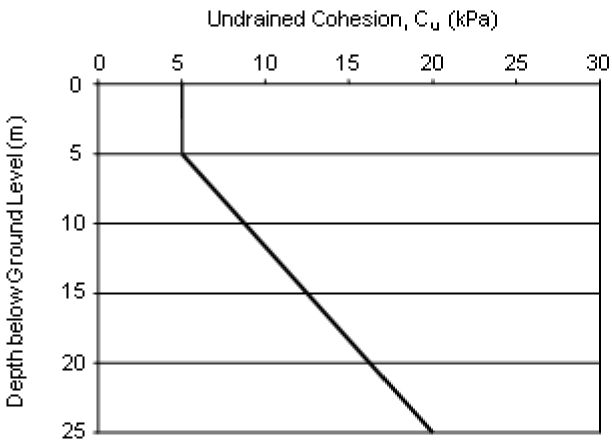


Figure 5.1
Estuarine (E)

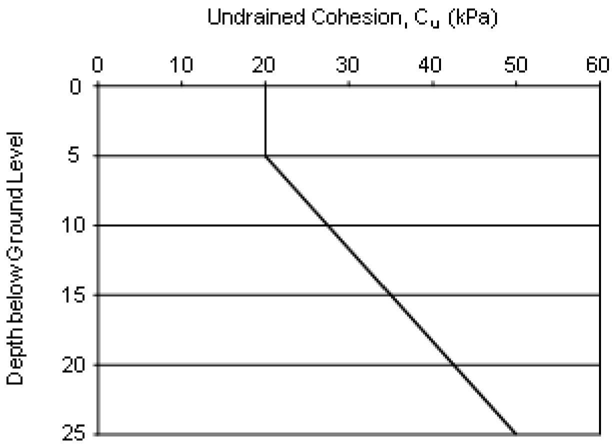


Figure 5.2
Fluvial Clay (F2)

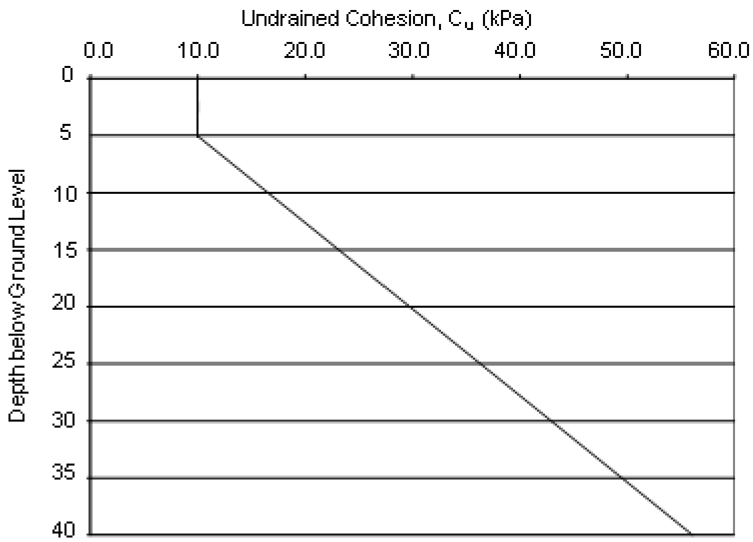


Figure 5.3
Marine Clay (M)

Note 1: Figures 5.1, 5.2 & 5.3 gives the undrained cohesion of the normally consolidated or slightly over-consolidated estuarine, fluvial and marine clay. For the areas under reclaimed land, the undrained cohesion of these clays shall be derived from in-situ tests, such as cone penetration tests or in-situ vane tests.

5.5 SOIL AND GROUNDWATER CHEMISTRY

Chemical tests shall be carried out during the site investigation for the Project to enable appropriate design considerations for durability. The results obtained shall be classified according to the corrosion properties of soil and groundwater given in SS EN 206 and SS 544. Any protective measures adopted shall comply with the recommendations of SS EN 206 and SS 544.

5.6 SITE INVESTIGATION

Additional site investigation may be conducted to justify any changes to the parameters given in Table 5.4 of this chapter as well as to obtain reliable information for an economic and safe design and to meet the tender and construction requirements.

The data collected shall be of sufficient quantity and quality to enable the following analysis to be carried out where appropriate:

a) All forms of constructions

- settlement estimates for effects of de-watering;
- evaluation of ground settlement due to expected permeability conditions;
- Structural deformation estimates due to effects of de-watering, increase or decrease in stresses around existing underground structures;
- evaluation of methods of building protection;
- evaluation of methods of ground treatment;
- stability and deformation analyses during and after construction works;
- identification of areas with potential problems.

b) Shallow and deep foundations

- ultimate and allowable bearing capacities for shallow foundations;
- ultimate and allowable vertical bearing capacities for deep foundations including the evaluation of negative skin friction;
- ultimate and allowable lateral bearing capacities for deep foundations;
- settlement estimates for shallow and deep foundations.

c) Underground structures

- stability and deformation analyses for temporary and permanent retaining structures;
- evaluation of bracing or anchoring system for temporary and permanent retaining structures;
- settlement estimates for bored tunnelling, including NATM;
- evaluation of tunnelling shield requirements;
- evaluation of tunnel face stability and protection;
- evaluation of uplift, buoyancy and floatation effects.

d) Earthworks and Soil Improvement Works

- effects of earthworks on ground water condition including, but not limiting to, water level, piezometric level and pore pressure changes;
- settlement and time estimates for improvement works.

e) Evaluation of the chemical corrosiveness of groundwater and soils and its effect on underground structures.

References

1. Anon (1970). The logging of rock cores for Engineering purposes. Engineering Group Working Party report, *Quarterly Journal of Engineering Geology 3, 1-24*

2. DSTA (2009). Geology of Singapore 2nd Edition, Defence Science Technology Agency.

3. Anon (1995). The description and classification of weathered rocks for engineering purposes. Quarterly Journal of Engineering Geology, 28, 207 - 242.

4. Clough, G.W. and Schmidt, B. (1981). Design and Performance of excavation and tunnels in soft clay. *Soft Clay Engineering. E.W. Brand and R.P. Brenner, ed. Elsevier, Amsterdam.569 - 636.*

5. Hoek E. and Brown, E.T. (1997) “Practical estimates of rock mass strength. *International Journal of Rock Mechanics and Mining Science, 34(8), 1165-1186.*

Appendix 5.1

Weathering classifications based on BS 5930 (2015)

The basis for assessing material scale weathering classifications for rocks in Singapore is based on BS 5930 (1999) which provided three different approaches for classifying material scale weathering in rocks. These are Approach 2 (for rocks that are moderately strong or stronger in a fresh state), Approach 4 (rocks that are moderately weak or weaker in a fresh state) or Approach 5 (special cases, such as karst). With the implementation of BS EN ISO 14689-1, BS 5930 (2015) clarified that whilst the weathering classifications based on Approach 2 may no longer be used for geotechnical category 2 projects due to conflict for strict compliance with that standard, this approach is still appropriate for geotechnical category 3 projects. Approach 4 and Approach 5 are still applicable in BS 5930 (2015).

Bukit Timah Granite and Gombak Norite

Approach 2 follows the system originally devised by Moye in 1955 for granite in Australia, and which has been used for many years in Hong Kong for the granite there. The igneous rocks of the Bukit Timah Granite and Gombak Norite should be described using Approach 2 for classification.

The Geotechnical Engineering Office (GEO) in Hong Kong has produced a list of simple indicators for the assessment of weathering grades in Hong Kong. A slightly amended version is given in Table 1. These indicators can also be used in the igneous rocks of Singapore, which have a similar strength when fresh.

It is particularly important to distinguish between grades III, IV, V and VI, (where present) as there is a significant difference in the engineering behaviour of each of these grades. Typically, the weathered granite in Singapore has a thick mantle of residual soil, with only limited underlying Grade V and/or Grade IV materials. In order to identify these during site investigation, careful and frequent sampling has to be carried out once the SPT value reaches 30 blows/300mm. The ‘Classifiers’ are not included in Table 1, as they are provided in BS 5930.

Table 1 – Bukit Timah Granite and Gombak Norite

Weathering classification for Bukit Timah Granite and Gombak Norite	
Grade	**Basis for assessment**
I	Intact strength, unaffected by weathering. Not broken easily by hammer – rings when struck. No visible discolouration.
II	Not broken easily by hammer – rings when struck. Fresh rock colours generally retained but stained near joint surfaces.
III	Cannot be broken by hand. Easily broken by hammer. Makes a dull or slight ringing sound when struck with hammer. Stained throughout.
IV	Core can be broken by hand. Does not slake in water. Completely discoloured.
V	Original rock texture preserved, can be crumbled by hand. Slakes in water. Completely discoloured.
VI	Original rock structure completely degraded to a soil, with none of the original fabric remains. Can be crumbled by hand.

Jurong Formation

The Jurong Formation includes a variety of sedimentary rocks that have been subjected to a variable degree of metamorphism. The Formation also includes the Pandan Limestone. The rocks of the Jurong Formation exhibit a wide range of strength in the Fresh State, and have weathered in different ways. The generally weak mudrock has weathered in a way that is best described by Approach 4, while the stronger sandstones and conglomerates have weathered in a way more like Approach 2. Where the rock is thinly bedded, which is the case in much of the Formation, it is considered impractical to apply different approaches. It is proposed to use Approach 2 for the Classification of the Jurong Formation wherever it is thinly bedded, and in all cases to Sandstone, Quartzite, Siltstone, Shale and Conglomerate. Where mudstone or Pandan Limestone is predominant in an area, then Approach 4 may be used for the mudstone and Approach 5 for the Limestone. The methods for assessing the weathering grade under Approach 2 are given in Table 2.

Table 2 - Jurong Formation

Weathering classification for Jurong Formation	
Grade	**Basis for assessment**
I	Intact strength, unaffected by weathering.
II	Slightly weakened, slight discolouration, particularly along joints.
III	Considerably weakened & discoloured, but larger pieces cannot be broken by hand. (RQD is generally >0, but RQD should not be used as the major criterion for assessment).
IV	Core can be broken by hand or consists of gravel size pieces. Generally, highly to very highly fractured, but majority of sample consists of lithorelics. (RQD generally = 0, but RQD should not be used as the major guide for assessment). For siltstone, shale, sandstone, quartzite and conglomerate, the slake test can be used to differentiate between Grade V (slakes) and Grade IV (does not slake).
V	Rock weathered down to soil-like material, but bedding intact. Material slakes in water.
VI	Rock degraded to a soil in which none of the original bedding remains.

For Approaches 4 and 5, reference can be made to BS 5930.

Old Alluvium

The Old Alluvium is an alluvial deposit that has been variably cemented, often to the extent that it has the strength of a very weak or weak rock. The upper zone of the Old Alluvium has typically been affected by weathering, and it is important that this weathering is described and classified. The use of Approach 4 of BS 5930 is recommended. However, although weathering of feldspars and mottling may be observed in borehole samples, it is generally difficult to assess the weathering grade from the visual inspection of samples obtained from boreholes. It is common practice in Singapore to use the blow count from SPT testing as an indicator of weathering classification. It must be understood that the SPT result is influenced by factors other than weathering. These other factors would include depositional environment, degree of cementation and the equipment and method used for the testing. Table 3 gives guidance on the typical SPT values for different weathering grades, although the final classification should be based on an assessment of both SPT and careful inspection of samples recovered. Where possible, correlation with the SPT values should be verified by comparing with any nearby large scale exposures of the Old Alluvium. Care must be taken in the SPT testing to ensure a representative test result, and classification should not be based on SPT alone.

It should be noted that at times, layers of hardpan can be found in the weathered Old Alluvium, and very high SPT values or refusal of penetration in the CPT may be due to hardpan rather than a change in weathering grade. Conversely, there may be a sudden drop in SPT N-value if a layer of uniform sand is encountered in a borehole which is not completely filled with stabilising fluid. Unlike the Bukit Timah Granite or the Jurong formation, there is little evidence of joints in the Old Alluvium. As a result, the weathering has typically penetrated as a discernible front from the surface. It is therefore unlikely that there will be more weathered beds under less weathered beds. Other factors, such as those given above, are likely to be the cause of a sudden drop in SPT resistance as the borehole is advanced.

Table 3 – Old Alluvium

Weathering Classification for Old Alluvium			
Class	**Classifier**	**Characteristics**	**Indicative SPT, Blows/300 mm***
A	Unweathered	Original strength	>50 (cannot usually be penetrated by CPTs with 20t load capacity)
B	Partially Weathered	Slightly reduced strength	
C	Distinctly Weathered	Further weakened	30 to 50
D	Destructured	Greatly weakened, of the mottled, bedding disturbed	10 to 30
E	Residual	No bedding remains	<10

* The SPT result should not be used in isolation to assess weathering, see text above

The tables given above for classifying weathered rocks provide the basis for establishing weathering descriptions under BS 5930 (1999, 2015). However, it is not necessarily the case that all of the weathering grades will be present at a particular location.

CHAPTER 6

FOUNDATIONS, PERMANENT RETAINING STRUCTURES AND EARTHWORKS

6.1 GENERAL

6.1.1 Scope

This Chapter covers the requirements for the design of foundations, permanent retaining structures and earthworks. The foundations, retaining structures and earthworks shall be designed for both short term and long term conditions.

6.1.2 Standards and Codes of Practice

6.1.2.1 All design shall comply with SS EN 1997, SS EN 1992, SS EN 1993, SS EN 1995 and their National Annexes as appropriate unless otherwise modified by subsequent sections of this Chapter.

6.1.2.2 The minimum strength class of concrete for all foundations shall be C32/40.

6.1.3 Ground Movements

6.1.3.1 Design of foundations, retaining structures and earthworks shall take into consideration all possible ground movements, ground conditions and groundwater levels.

6.1.3.2 The design shall ensure that movements are within limits without affecting the function, durability and aesthetic value of the Works.

6.1.3.3 It shall be demonstrated that the chosen foundation system or form of construction shall not result in excessive settlement of adjoining properties, structures or utilities as specified in Chapter 20.

6.1.4 Deleterious Substances in Soils

Substances in soils and ground water, which are potentially deleterious to materials used in structures in contact with the ground, shall be considered in the design and specification of all such structures and any protection system.

6.1.5 Combining Various Foundation Types in a Single Structure

A combination of different foundation types or systems within a single structure shall not be permitted.

6.2 FOUNDATIONS

6.2.1 Spread Foundations

6.2.1.1 General

6.2.1.1.1 Spread foundations such as pads, strips and rafts may be used where there is a suitable bearing stratum at founding level. These shall include the base slabs of cut and cover tunnels and stations where the base slab is not wholly supported by Deep Foundation Elements (DFEs).

6.2.1.1.2 Where compressible or loose soil layers occur below the founding stratum, deep raft foundations on DFEs shall be used.

6.2.1.2 Settlement

6.2.1.2.1 The settlement should be designed according to serviceability limit state and the movement limits of railway, road and any other structure.

6.2.1.2.2 For deep raft foundations, settlement analyses shall be carried out by using either Finite Element (FE) or Finite Difference (FD) methods.

6.2.1.2.3 Where values for the subgrade modulus are used in the calculations, the values shall be confirmed by FE or FD analyses for an appropriate range of foundation geometries, subject to verification of the ground condition on site.

6.2.1.2.4 In addition to elastic settlements, consideration shall be given to the potential for non-elastic settlements such as those due to creep and consolidation.

6.2.1.3 Bearing resistance

The spread foundations shall satisfy the ultimate limit states for bearing resistance.

6.2.1.4 Groundwater Level

The groundwater level shall be considered to be at finished ground level for assessing the allowable bearing capacity of a spread foundation.

6.2.1.5 Influence of Adjacent Foundations

Where the pressure beneath a foundation is influenced by adjacent foundations, the effect of combined bearing pressure shall not exceed the bearing resistance, and maximum allowable differential settlements shall not be exceeded.

6.2.1.6 Foundations on Slopes

For spread foundations on or near slopes the following additional analyses are required:

a) Stability of the slope
b) Bearing resistance of the foundation on the slope

6.2.1.7 Downdrag (Negative Skin Friction)

6.2.1.7.1 For deep raft foundations, downdrag on the structure above the founding level shall be considered for marine soils, alluvial soils, estuarine soils, man-made ground and any other material that is prone to consolidation or is to be consolidated.

6.2.1.7.2 Downdrag of the whole foundation system due to groundwater lowering shall be considered.

6.2.1.7.3 Computation of downdrag shall be by effective stress analysis.

6.2.2 Deep Foundation Elements (DFEs)

6.2.2.1 General

6.2.2.1.1 The term Deep Foundation Element (DFE) shall include all foundation elements designed to transfer loads by shaft friction and/or end bearing. These elements shall include but not limited to piles, pile groups, diaphragm walls, barrettes, secant and contiguous bored pile walls. In the event of any uncertainty as to whether or not a particular foundation type or element is covered by this definition written clarification shall be sought from the Engineer.

6.2.2.1.2 When choosing the type of foundation, consideration shall be given to the impact of noise and vibration during DFE installation and current legislation on the use of piling and other construction equipment.

6.2.2.1.3 The minimum cover to all reinforcement (including links) shall be 75mm.

6.2.2.1.4 In the design of driven piles, stresses arising from impact and shock from piling hammers shall be considered.

6.2.2.2 Downdrag (Negative Skin Friction)

6.2.2.2.1 Downdrag shall be considered in the design in marine soils, alluvial soils, estuarine soils, made ground and any other material that is prone to consolidation or is to be consolidated.

6.2.2.2.2 Downdrag of the whole foundation system due to groundwater lowering shall be considered.

6.2.2.2.3 Computation of downdrag shall be by effective stress analysis.

6.2.2.2.4 Raking DFEs shall not be used in areas where negative skin friction is anticipated.

6.2.2.3 Representative Loads

The representative loads of the DFE shall be the greater of:

a) Applied dead load + downdrag and non-transient imposed load (eg. Fixed plant and equipment in stations);
b) Applied dead load + total imposed load.

6.2.2.4 Lateral Loads

6.2.2.4.1 Laterally loaded DFEs shall satisfy the requirements given in Table 6.1. The analysis shall use “p-y” curve, finite element or finite difference.

6.2.2.4.2 For lateral loading, maximum allowable deflections shall be calculated at pile cut-off level.

Table 6.1 – Maximum Horizontal Deflection

Foundation Type	**Maximum Allowable Horizontal Deflection**		
	Total		**Differential**
	Short Term	Long Term	Short & Long Term
Laterally Loaded DFEs	15 mm	25 mm	1:1000

6.2.2.5 Compressive Ground Resistance of DFE

6.2.2.5.1 The design of the DFE shall be demonstrated to be sufficient to provide the required compressive ground resistance in ULS and meet the settlement criteria. Total stress analysis and effective stress analysis shall be carried out and whichever is more onerous shall be considered in the design.

6.2.2.5.2 The compressive ground resistance of DFE shall be designed in accordance with SS EN 1997 and its National Annex.

6.2.2.5.3 In addition, a check on characteristic shaft friction alone shall be carried out such that the ratio of characteristic shaft friction resistance to the representative load shall not be less than 1.5 except in any of the following cases:

Case a. When the DFE has been installed by driving.

Case b. Where there is safe man access to the base of the DFE, any loose or remoulded material is removed from the base, and the base inspected and confirmed dry before casting the concrete.

Case c. If the DFE is base grouted, using a proven method of base grouting to the acceptance of the Engineer.

Case d. Where the end bearing is provided by S1 (SI or SII) or G1 (GI or GII) rock, and toe coring is carried out to confirm the pile/rock contact for every DFE.

If the design is based on any of these four cases, the appropriate requirements for driving, access and inspection, base grouting or toe coring must be included on the loading plan drawings.

6.2.2.5.4 The DFE design shall be verified by Ultimate Load Testing.

6.2.2.5.5 For the load combination where the water table is taken at the underside of the lowest base slab as defined in Chapter 8, the geotechnical factors of safety above for DFEs may be reduced by 20%, provided that the structure is designed for the associated settlement and differential settlement.

6.2.2.6 DFE Interaction

The group effect of DFEs shall be checked in settlement and lateral deflection and in ground resistance. The interaction between DFEs shall be assessed and considered in the calculation of the capacity and settlement based on Poulos and Davis (1980) or any other methods accepted by the Engineer.

6.2.2.7 Ground Tensile Resistance of DFE

For driven piles the ultimate skin friction under tension shall be taken as no more than 75% of the ultimate friction measured in compression. No reduction factor is required for bored piles.

6.2.2.8 The ground tensile resistance of DFE shall be designed in accordance with SS EN 1997-1 and its National Annex. Furthermore, the ground tensile resistance shall be divided by an additional model factor of 1.15.

6.2.2.9 For flyover/viaducts, no pile foundation shall be subjected to tension permanently.

6.2.2.10 DFE groups under tension loading shall be checked for:

a) The sum of the uplift resistance of the individual DFEs, allowing for interaction effects.

b) The sum of the shear resistance along the perimeter of the group and the effective weight of the soil and DFEs within the perimeter.

6.2.2.11 All structural connections shall be designed for the design tensile force in accordance with SS EN 1992 and SS EN 1993.

6.2.2.12 DFE Design Requirements

6.2.2.12.1 The structural design of DFEs shall meet the following requirements:

a) Compressive Axial Load

Concrete DFEs subject to axial load shall be designed such that the average compressive stresses across the whole section of the DFE at serviceability limit state shall not exceed the lesser of 0.25 times the characteristic cube strength of concrete at any point along the DFE or the value in accordance with SS EN 1992. Where permanent casing is provided the allowable compressive stress may be increased, subject to the acceptance of the Engineer.

b) Axial Load with Coexistent Lateral Load

Concrete DFEs shall be designed as short columns according to SS EN 1992 and checked at both the ultimate and serviceability limit states. The DFEs shall be designed such that the maximum crack width under representative load is 0.25mm.

c) Pure Tensile Axial Load

The safe tensile axial working load of the DFE shall be determined by multiplying the cross-sectional area of steel reinforcement with the permissible tensile stress. The permissible tensile stress under representative load shall not exceed $0.58f_y$ N/mm^2. In addition, the DFEs shall be designed such that the maximum crack width under representative load is 0.25mm. The crack width calculation shall be according to SS EN 1992. The stiffening effect of concrete shall not be considered in the calculation. The DFE shall be reinforced to the depth necessary to mobilise the required ultimate skin friction capacity in tension. Laps shall be avoided wherever possible. Where laps are necessary they shall be full strength laps assuming the reinforcement is working at the ultimate limit state at full design strength (i.e. characteristic strength of reinforcement divided by the partial safety factor for reinforcement).

6.2.2.12.2 In general, bored piles shall not be constructed without steel reinforcement throughout. The reinforcement provided shall not be less than the requirement in SS EN 1992 Clause 9.8.5.

6.2.2.12.3 Where the durability assessment, as required in Design Criteria Chapter 1, demonstrates that durability of the DFE is of concern, then suitable measures shall be taken to improve the durability. Measures to be considered shall include one or more of the following: increased concrete cover, permanent protective outer casing (e.g. HDPE), protection to the reinforcement, cathodic protection, or other suitable measures.

6.3 SETTLEMENT/HEAVE

6.3.1 The limits given for settlement and differential settlement shall also be complied with in terms of heave and differential heave.

6.3.2 The maximum allowable foundation settlement shall be in accordance with Table 6.2, unless otherwise accepted by the Engineer.

Table 6.2 – Maximum Allowable Settlements

Foundation Type	**Maximum Allowable Settlement**		
	Total		**Differential**
	Short Term	Long Term	Short & Long Term
Spread Foundation	20 mm*	20 mm*	1:1000*
Deep Foundation Element	15 mm	25 mm	1:1000

The limits in Table 6.2 marked with an asterisk (*), may be exceeded if it is demonstrated that the structure is designed for the movements to which it will be subjected without compromising the serviceability of the system.

6.3.3 Short-term settlement shall consist of immediate non-recoverable settlement, elastic settlement and consolidation settlement which occurs between the date of casting and the date of Substantial Completion of that part of the works.

6.3.4 Long-term settlement shall consist of immediate non-recoverable settlement, elastic settlement, consolidation settlement and creep settlement from the date of casting to the end of the design life.

6.3.5 Settlement shall be the settlement occurring from the time at which the base slab/pile cap is cast and shall be measured at the structural surface of the base slab/pile cap.

6.3.6 Railway Track

6.3.6.1 Where the foundations provide support to structures carrying railway track, the maximum anticipated settlement and differential settlement of the railway track shall be calculated. The settlement calculation shall include the effects of all anticipated loads and effects occurring after the track has been laid. These shall include but not be limited to imposed loads, any dead loads not applied prior to track laying, groundwater recovery, downdrag, known future development loads (including dewatering during such developments), creep of the material forming the foundation, and creep and/or consolidation of the founding strata. The design shall ensure that the settlement of the track under these loads and effects does not exceed 15mm and that differential settlement of the track or its plinth does not exceed 1:1000 in any plane

6.3.6.2 Settlement is measured at any location along the track as the change of the level of the track (where the track level is taken as the mean level of the two rails). Differential settlement is measured in two directions, as follows:

a) Perpendicular to the track – 1:1000 between the rail heads measured as the difference in the change of levels between the two rails (mm) at that location divided by a nominal gauge of 1435 mm.

b) Parallel to the track – 1:1000 between any two points 3 metres apart, measured as the difference in the change of levels between the two points (mm) divided by 3000 mm.

6.3.6.3 The assumptions with respect to backfill and recovery of the water table level before the track is laid shall be justified and shall be stated on the drawings.

6.3.7 Road

Where the foundations provide support to a road, or structures carrying a road, the maximum anticipated settlement and differential settlement of the road shall be calculated. The design shall ensure that the settlement and differential settlement do not adversely affect the function of and maintenance requirements for the road. Consideration of differential settlement shall be given to all aspects of the road, including, but not limited to the road pavement, culverts, drains, sumps, sewer manholes, ancillary structures, mechanical and electrical plant, cabling and services.

6.4 DEBONDING OF DFEs

6.4.1 For DFEs to be constructed for developments adjacent to permanent works, they shall be debonded such that there is no load transfer to the permanent works.

6.4.2 Alternatively, the permanent works shall be designed for imposed load from adjacent development. The assumptions for load transfer along the length and at the base of the DFEs shall be verified by means of instrumented load tests. The number of instrumented load tests must be sufficient to provide information for all types of DFEs in all ground conditions. Any imposed loads from the DFEs shall be additional to the loads and surcharges defined in Chapter 3 and in the Particular Specification.

6.5 LOAD TESTING

6.5.1 General

6.5.1.1 The number and type of load tests shall be specified in accordance with the requirements given in Table 6.3. The number and type of tests shall be indicated on the Drawings

Table 6.3 – Minimum Requirements for Testing of Deep Foundation Elements

		Type of Load Test		
Type of Structure		**Ultimate Load Test (Vertical, Horizontal and Tension respectively)**	**Working Load Test (Vertical, Horizontal and Tension respectively)**	**Pile Integrity Test**
Major Above-Ground infrastructures - Viaducts		1 per pile type[1] and pile size[2], for each representative ground condition	1 for every 10 piers	1 for every 5 piers
Major Underground Infrastructures – Tunnels / Stations	**Piles**	1 per pile type[1] and pile size[2], for each representative ground condition	2% or 2 numbers whichever is greater	3% or 3 numbers whichever is greater
	Barrettes	1 per pile type[1] and pile size[2], for each representative ground condition	Not required[3]	3% or 3 numbers whichever is greater
	Load bearing diaphragm walls and secant piled walls	1 in every 400m for each representative ground condition	Not required	1 in every 200m

Table 6.3 – Minimum Requirements for Testing of Deep Foundation Elements (Cont'd)

	Type of Load Test		
Type of Structure	**Ultimate Load Test (Vertical, Horizontal and Tension respectively)**	**Working Load Test (Vertical, Horizontal and Tension respectively)**	**Pile Integrity Test**
Buildings / Above-Ground Stations	1 per pile type[1] and pile size[2]	1% or 1 number whichever is greater	2% or 2 numbers whichever is greater
Pedestrian Overhead Bridges	Not required	1 per bridge	Not required

Note:

1. The pile types are differentiated by their effects on the soil and the installation techniques, such as bored cast-in-place concrete piles installed by boring, driven precast piles installed by jack-in method, driven steel bearing piles installed by a drop hammer, etc.
2. The pile size can be considered representative for working piles up to twice the diameter of the test piles.
3. Working load test of 2 numbers or 1% of total number of working piles or 1 for every 50 metres length of proposed building, whichever is greater, shall be carried out on barrette piles which are supporting buildings of 10-storeys or more.
4. For the purpose of establishing the number of tests required, the percentages given in Table 6.3 relate to the total number of working DFEs of the type of structure. Calculations of the number of tests as a percentage of the total number of DFEs shall be made to one decimal place. If the result is not a whole number then the value shall be rounded up to the nearest whole number: e.g., if 116 working piles were installed for the major infrastructure – tunnels founded on piles, the 2% requirement would be for 3 working load tests.

6.5.1.2 Where more than 400 DFEs are installed in similar ground conditions, using similar methods, on a single type of structure by the same piling contractor then the number of tests can be reduced on the following basis: -

a) For the first 400 DFEs, the percentage shall be as shown in the Table 6.3.

b) For DFEs in excess of the first 400, the percentage tested may be reduced to half of that shown in the Table 6.3 for the relevant tests.

6.5.1.3 Table 6.4 provides guidance on various acceptable DFE test methods. The allocation of the type and number of test methods to meet the requirement of various load tests in Table 6.3 shall be decided by the Engineer, e.g. if 3 working load tests are required for the major infrastructure – tunnels founded on piles, the Engineer may specify to conduct 2 static load test (Kentledge) and 1 dynamic load test.

Table 6.4: Guidance on Various Acceptable DFE Test Methods

Type of Load Test	List of Acceptance Pile Test Methods
Ultimate Load Test	• Static load test (Kentledge)[1] • Static load test (Reaction piles/anchors) • Bi-directional static load test (O-cell)[2]
Working Load Test	• Static load test (Kentledge)[1] • Static load test (Reaction piles/anchors)[3] • Dynamic load test (DPT/PDA with CAPWAP)[4] • Quasi-static load test (Statnamic)[5]
Pile Integrity Test	• Dynamic load test (DPT/PDA with CAPWAP) • Low strain impact test • Sonic coring • Sonic logging

Note:

1. For safety, not more than 3000 tonnes of concrete blocks shall be used.

2. The derivation of load-displacement curves shall be in accordance with SS EN 1997-1.

3. Working piles shall not be used as reaction piles.

4. Dynamic load test shall be calibrated against static load test on the same pile type, of similar length and cross section, and in comparable ground conditions.

5. Quasi-static load test shall be interpreted by suitably qualified personnel, and may also be used for ultimate load test subject to the Engineer's acceptance.

6.5.1.4 For piles using characteristic unit end bearing resistance more than 10MPa in design, interface coring test shall be conducted at the pile base for 5% of the total number of working piles, which can be reduced to 2% if base grouting is carried out.

6.5.1.5 Pile profile test to assess verticality and shapes of pile bore prior to concreting shall be carried out, for working piles with diameter bigger than 1.8m which either have no steel casing going through soft or loose soil during construction or have no redundancy.

6.5.2 Ultimate Load Tests

6.5.2.1 Test Loads

6.5.2.1.1 The target for the ultimate load testing is to achieve the Ultimate Limit State (ULS) criterion.

6.5.2.1.2 The ULS criterion for an ultimate load test is when the settlement of the pile head has exceeded 10% of the pile diameter.

6.5.2.2 Dimensions for Ultimate Load Tests of DFEs

6.5.2.2.1 Ultimate load tests on piles can be considered representative for working piles up to twice the diameter of the ultimate test piles, provided that they are installed in similar ground conditions.

6.5.2.2.2 For the load testing of barrettes and diaphragm wall, the contractor may propose the testing of piles in place of the barrettes or diaphragm wall. However, the contractor must ensure that the diameter of the pile is at least equal to the minimum dimension of the diaphragm wall or barrette and that the method of construction for the pile be similar to that used for construction of the diaphragm wall or barrettes.

6.5.2.3 Ground Conditions

Ultimate test DFEs shall be installed in ground similar to that where the working DFEs are to be installed.

6.5.3 Working Load Tests

6.5.3.1 Test Loads

The test loads in a Working Load Test shall be 150% of the representative load of the DFEs.

6.5.3.2 Serviceability Limit State Criteria

6.5.3.2.1 The Serviceability Limit State (SLS) criteria shall be established at 1.0 and 1.5 times the representative load. An allowable residual settlement upon final release of test load shall be determined. These criteria shall be calculated to be consistent with the whole foundation system meeting the settlement requirements under Table 6.2 and shall be used as the criteria for assessing load tests. These criteria shall be stated on the Drawings.

6.5.3.2.2 Similar SLS criteria shall also be established for tension DFEs. SLS criteria at 1.0 times the representative load shall be established for laterally loaded DFEs.

6.5.3.2.3 For guidance, some typical failure criteria are given below. The maximum settlement figures may be increased by 2.2% for every metre length of DFE beyond 30 metres.

6.5.3.2.4 SLS Failure Criteria for Bored Piles

a) The maximum settlement at representative load (second cycle) exceeds 7mm, and

b) The maximum settlement at 150% representative load exceeds 14mm.

6.5.3.2.5 SLS Failure Criteria for Driven Piles

a) The maximum settlement at representative load (second cycle) exceeds 9mm, and

b) The maximum settlement at 150% representative load exceeds 20mm.

6.5.3.2.6 Failure Criteria for all DFEs

a) Failure of the DFE materials (due to defects in the DFEs).

b) Failure of concrete to reach the design compressive strength.

c) The residual settlement upon the final release of the load exceeds 5mm.

6.5.3.3 Failure of Working Load Test

6.5.3.3.1 A DFE is considered to have failed a loading test if it does not comply with either of the SLS criteria. Where a pile fails the test the Contractor shall install suitable compensation piles. If, in the opinion of the Engineer, it is impracticable or inadvisable to install compensation piles in place of a failed pile, the Contractor shall submit proposals to rectify the defect. The proposal is subject to the acceptance of the Engineer.

6.5.3.3.2 Working load tests shall be in the direction of the design load, i.e., Piles where the major load is in tension shall be tested in tension.

6.5.4 Selection of DFEs for testing

DFEs to be tested shall be proposed for the acceptance of the Engineer. Piles for testing shall be selected after they have been constructed unless the method of testing requires the piles to be pre-selected

6.6 PERMANENT GRAVITY AND CANTILEVER RETAINING WALLS

6.6.1 Lateral Earth Pressures

6.6.1.1 In computing the lateral pressure of the retained earth, the appropriate coefficient of earth pressure shall be used.

6.6.1.2 For cohesive soils, the total design pressure shall not be less than that of an equivalent height of water pressure.

6.6.2 Water Pressure

Water pressure must be considered in the design notwithstanding that weep holes are provided. Design water level shall be taken at finished ground level.

6.6.3 Overall Stability

6.6.3.1 The overall stability shall be verified as required in SS EN 1997.

6.6.3.2 In addition, contribution of passive resistance of the top 1500mm of the ground in front of wall shall be ignored for permanent retaining walls.

6.6.3.3 For abutments higher than the surrounding ground, the design water level shall be taken at 1/3 the height of the abutment measured from the surrounding ground. Additional checks shall also be carried out for the design water level to be considered at the top of the abutment. In such case, the design value of the effect of actions (E_d) may be multiplied with a model factor $\gamma_{S;d}$ of 0.75.

6.6.4 Use of DFEs for Retaining Structures

Where DFEs are used, the interface friction angle between the base of the wall and the underlying soil shall be taken as zero.

6.6.5 Settlement and Deflections

The design shall include an assessment of the deflections and settlement of the retaining structure. The design shall demonstrate that the anticipated settlement and deflection will not cause damage to the retaining wall or to adjacent structures or utilities.

6.6.6 Seepage

Where appropriate, seepage around or under the structure shall be considered when calculating the earth pressures generated on both sides of the retaining structure.

6.7 EARTHWORKS

6.7.1 General

6.7.1.1 For the design of slopes, the effects of ground deterioration due to local climatic conditions shall be considered in the analysis. The steepest unreinforced permanent slope allowed shall be 2.0 horizontal to 1.0 vertical, unless otherwise accepted by the Engineer.

6.7.1.2 For drainage, an open concrete-lined channel should be used whenever possible. The design shall demonstrate that the short term and long term behaviour of the soils will not in any way affect the functional and aesthetic value of all earthworks. The design shall allow for all planned excavations at the toes of the slopes including the construction of drains in addition to the allowance for unplanned excavation as required in SS EN 1997.

6.7.2 Stability of Earthwork and Cut Slopes

6.7.2.1 The stability of all earthwork and cut slopes shall be verified as required in SS EN 1997.

6.7.2.2 In addition, the ratio of the characteristic resistance to the effect of representative actions shall be at least 1.5.

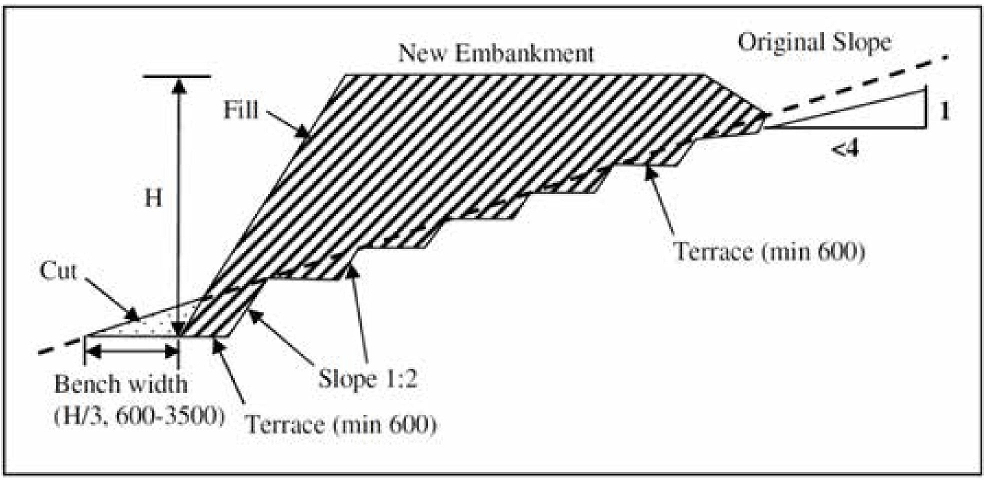
6.7.3 Embankment

6.7.3.1 Embankment shall not be used for railway tracks.

6.7.3.2 The design shall include benching, toe keys, ditches and drainage as required and provision for clearing vegetation, digging and breaking up of topsoil, and prevention of slope erosion.

6.7.3.3 For embankments on existing slopes (Figure 6.1) which exceed 1 vertical to 4 horizontal, a horizontal bench shall be cut to support the toe. The minimum width of the bench shall be equal to 1/3 of the vertical height of the fill (measured from the toe) but the bench width shall be within 0.6m to 3.5m. The existing slopes shall be designed to have horizontal terraces cut over the full area to be covered by new fill. Each horizontal terrace shall be at least 600mm in width and shall be stepped in slopes no greater than 1 vertical to 2 horizontal.

Figure 6.1 Embankments on Hillsides



6.7.3.4 The design drawings shall indicate the construction sequence such that horizontal terraces are cut progressively as each embankment layer is placed.

6.7.4 Soil Improvement

6.7.4.1 Soil Improvement methods may be included either singly or as a combination of methods to meet the design requirement and performance specification.

6.7.4.2 Pilot areas of soil improvement as agreed with the Engineer shall be carried out to verify the design prior to the commencement of the main soil improvement work.

6.7.5 Drainage

Provision shall be made in the design for an adequate drainage system that incorporates sufficient capacity for the design rainfall run-off based on Public Utilities Board (PUB)'s Code of Practice for Surface Water Drainage. The anticipated volume of run-off shall be determined. Consideration shall be given to ensure the long-term performance of the drainage system.

6.7.6 Non-Suspended Apron Structures and Services

6.7.6.1 Settlement of non-suspended apron structures and services shall be designed to be within the limits which shall not in any way affect the function, durability and aesthetic value of such structures and services.

6.7.6.2 Special attention is required where the apron structures and services are over compressible materials e.g. Marine Clays, Estuarine Clays and are adjacent to structures on DFEs or deep rafts.

6.7.7 Reinforced Earth Slopes and Walls

6.7.7.1 The design of reinforced earth slopes and walls shall satisfy both long term and short term conditions. The partial factors of safety in BS 8006 shall be complied with.

6.7.7.2 The reinforced earth slopes or walls shall be checked for both internal and external stability in accordance with BS 8006. Loading shall be in accordance with Chapter 3.

6.7.7.3 The strength class of concrete surfacing panels for reinforced earth walls shall be at least C32/40 with a minimum concrete cover of 40mm. The panels shall be designed for vehicular impact load in accordance with Chapter 3 or shall be designed such that they can be replaced. As part of the design, a method statement shall be prepared for replacement of the panels.

6.7.7.4 Allowance shall be made in the design for corrosion of metal strips used as reinforcement as follows:

$$d_c = d_{design} + r_c * (\text{life span of structure})$$

Where,

d_c = actual thickness or diameter of reinforcing material to be used in construction (mm)

d_{design} = thickness or diameter of reinforcing material determined from design calculation (mm)

r_c = rate of corrosion (mm/yr)

6.8 TRANSITION SLABS

6.8.1 General

To provide a smooth transition from at-grade sections to elevated and underground sections, a transition slab shall be provided. The transition slab shall meet the following requirements for road and rail structures.

6.8.2 Transition Slab for Highway Bridges

The transition slab shall be of a length such that in the period between its completion (as determined by the Engineer) and the end of the Maintenance Period, the change in gradient between the transition slab and road/bridge shall be less than 0.25% but in any event its length shall not be less than 6m. If necessary, ground improvement works shall be specified in conjunction with the approach slab to meet this requirement.

6.8.3 Transition Slab for Railways

6.8.3.1 Design calculations shall be carried out to estimate the anticipated settlements and rotations of transition slabs and it shall be shown that these movements shall not in any way affect the performance of the railway line.

6.8.3.2 The length of transition slab shall be calculated and in no case be less than 6m nor less than that given by the following:

$$\mathrm{L} = 1.5\,\mathrm{H}\ \tan(45° - {}^{1}/_{2}\,\varnothing)$$

Where,

L = minimum length of transition slab from centre of slab support

H = vertical distance from bottom of transition slab to bottom of abutment

Ø = Angle of internal friction of backfill beneath slab, in degrees

6.8.3.3 The transition slab shall be designed assuming that it receives no support from the backfill for a distance not less than 4m nor less than $\mathrm{H}\ \tan(45° - {}^{1}/_{2}\,\varnothing)$ from the back of the abutment.

6.8.4 Use of Finite Element or Finite Difference Modelling Techniques

6.8.4.1 The requirements on the use of finite element (FE) or finite difference (FD) modelling techniques as specified in Chapter 16 shall be followed for permanent works design.

6.8.4.2 Where structural elements have been used in a FE or FD model, justifications must be provided on their material properties and connection details. For permanent structural elements, the full stiffness of elements shall be used in the model.

CHAPTER 7

BORED TUNNELS AND RELATED WORKS

7.1 SCOPE

This chapter covers the design requirements for the following structures:

a) Segmental linings (permanent & temporary);
b) Cast in-situ linings (permanent & temporary);
c) Sprayed concrete linings (SCL);
d) Support system in rock;
e) Cross passageways; and
f) Escape shaft.

7.2 STANDARDS

The design of bored tunnels and related works shall comply with SS EN 1990, SS EN 1992, SS EN 1993, SS EN 1997, other relevant codes and the additional requirements herein.

7.3 DEFINITIONS OF SOFT GROUND AND ROCK

7.3.1 Soft ground shall include all grounds except GI, GII, SI & SII (see Chapter 5, Geotechnical Parameters).

7.3.2 Rock shall include grounds GI, GII, SI & SII (see Chapter 5, Geotechnical Parameters).

7.4 ACTIONS

7.4.1 Combinations of Actions

The envelopes of the stress resultants at both ultimate and serviceability limit states shall be determined from the critical Combinations of Actions listed in Table 7.1.

Table 7.1 – Critical Combinations of Actions for design of bored tunnels

Combinations of Actions	**Description**
Combination 1	Full ground overburden pressure using Ground Water Level (GWL) at lowest credible level together with (where more onerous) imposed surcharge load
Combination 2	Full ground overburden pressure using GWL at finished Ground Level (GL) together with (where more onerous) imposed surcharge load
Combination 3	Full ground overburden pressure using Design Flood Level
Combination 4	Full ground overburden pressure using GWL at lowest credible level together with (where more onerous) imposed surcharge load, and additional distortion of +/-15mm on diameter to allow for future development
Any other more onerous combination of actions	

7.4.2 Distortional Loading Coefficient

7.4.2.1 Distortional loading due to out-of-balance vertical and horizontal earth pressures shall be used in the analysis of tunnel in soft ground. The design shall satisfy the distortional loading coefficient, K, which is the ratio of horizontal earth pressure to vertical earth pressure prior to lining deformation as shown in Table 7.2. A sensitivity study shall be carried out to demonstrate that design is adequate and covers all situations.

Table 7.2 – Distortional loading coefficient (K) for design of bored tunnels

Soil Type	**K**
E, M, F2	0.75
B, O, GV, GVI, F1	0.5
SV, SVI	0.4
SIII, SIV, FC, GIII, GIV	0.3

7.4.2.2 The distortional loading on the linings shall take account of the short-term and long-term of horizontal and vertical earth pressures.

7.5 DESIGN CONSIDERATIONS AND REQUIREMENTS

7.5.1 General

The design of the tunnel linings shall take into account the required life span, the proposed use, the ground conditions, proximity of one tunnel to the other, the proximity of adjacent structures and the method, sequence and timing of construction

7.5.2 Tunnel Size

7.5.2.1 The theoretical internal tunnel size shall be of sufficient size to accommodate all operational envelope requirements and provision for services, fittings and furniture as required. These may include but are not limited to the following: -

a) Structure gauge in rail tunnels with full allowance for the maximum cant;
b) Track support system in rail tunnels;
c) Plant and all associated furniture, bracket, cabling & etc.;
d) Traction supplies;
e) Maintenance requirements such as walkway and its envelope;
f) Vertical and lateral clearance envelope in road tunnels, with full allowance for the super-elevation and curve widening to take into account stopping sight distance;
g) Dry riser fire-main with hydrants;
h) Ventilation equipment and ducting;
i) Drainage channels and pipes;
j) Lighting;
k) Pumping main;
l) Signage; and
m) Adequate blockage ratio to satisfactorily minimize the piston effect of the trains or road vehicles as appropriate. (If a non-circular tunnel is selected, it shall provide the same blockage ratio as a circular tunnel)

7.5.2.2 A minimum space of 100 mm (on radius) all around shall be provided in the design to accommodate: -

a) Construction tolerances given in the Materials & Workmanship (M&W) Specification;
b) The deformations of the tunnel section under design actions; and
c) Future movements (e.g. due to future development).

7.5.3 Design Methods

7.5.3.1 The analyses or design methods for bored tunnel linings in soft ground shall take into account interaction between lining and ground, deflection of lining and re-distribution of loading dependent upon the relative flexibility of lining and compressibility of ground.

7.5.3.2 Acceptable methods include: -

a) Continuum model by AM Muir Wood(1) combined with discussion by DJ Curtis(2)

b) Bedded beam model as Duddeck and Erdmann(3)

c) Finite element model

7.5.3.3 Due account shall be taken of the degree of flexibility of the linings to be used in the soft marine clays and fluvial deposits. The flexibility may have to be reduced in order to maintain acceptable values for the deflection of the lining.

7.5.3.4 The design methods and analyses of the tunnel linings in rock shall take into account the various types of material to be encountered.

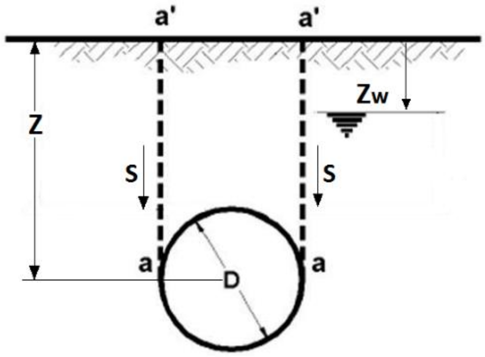
7.5.3.5 The design of tunnel linings in rock shall take into consideration the methods of construction appropriate to the rock types. The shape, length, depth and width of excavations shall be limited in the construction method to suit the material encountered.

7.5.3.6 In the evaluation of the design rock loads for both the temporary and permanent tunnel linings, the proposed method of construction shall be taken into consideration

7.5.4 Uplift and Heave

7.5.4.1 Tunnels shall be checked for the possibility of losing equilibrium due to uplift by water pressure (buoyancy) with the following method (Figure 7.1). The self-weight of soil type Estuarine (E), shall be divided by a model factor $\gamma_{S;d}$ of 1.2, when it is a stabilizing action.

Figure 7.1 – Verification of Uplift Limit State (UPL)



UPL limit state: $\gamma_E \, G_{dst;k} \leq \gamma_E \, G_{stb;k} + R_k/\gamma_R$

Uplift, $G_{dst;k} = \gamma_w \, {}^{\pi D^2}/_4$

Stabilizing action, $G_{stb;k} = W + \gamma' D\left(Z - Z_w - {}^{\pi D}/_8\right) + \gamma_{bulk} D \, Z_w$

Resistance, $R_k = 2sZ$

Where, γ_E = partial factor for the effect of an action

γ_R = partial factor for a resistance

γ_w = weight density of water

W = self-weight of tunnel

D = outer diameter of tunnel

γ' = effective weight density of soil

γ_{bulk} = bulk weight density of soil

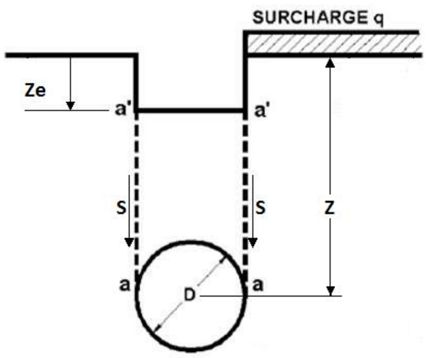
s = average shear resistance along a-a’

Note: For non-circular tunnels, the flotation check shall be derived from above principle

7.5.4.2 For shallow bored tunnels in clay, they shall also be checked for the possibility of basal heave due to shear failure of the ground at tunnel invert level by Bjerrum and Eide[(11)] method (Figure 7.2). The resistance shall be divided by a model factor $\gamma_{R;d}$ of 1.5 for this check. The self-weight of soil type Estuarine (E), shall be multiplied with a model factor $\gamma_{S;d}$ of 1.2, when it is a destabilizing action.

Figure 7.2 – Verification of Geotechnical Limit State (GEO) due to Shear Failure at Tunnel Invert Level

For the general case:



GEO limit state: $\gamma_E\ E_{dst;k} - \gamma_E\ E_{stb;k} \leq R_k/\gamma_R$

Destabilizing action, $E_{dst;k} = \gamma_{b1}\ {}^{\pi D}/_{4} + \gamma_{b2}\ Z_e + q$

Stabilizing action, $E_{stb;k} = W/D$

Resistance, $R_k = N_c c_u + \ 2s_u(Z - Z_e)/D$

Where, γ_E = partial factor for the effect of an action

γ_R = partial factor for a resistance

N_c = bearing capacity factor

c_u = average undrained shear strength of soil in the zone of the tunnel invert

γ_{b1} = average bulk weight density of soil in zone of tunnel

γ_{b2} = average bulk weight density of soil over depth Z_e

Z = depth to tunnel axis

Z_e = depth of excavation above tunnel (if any)

Q = variable action at ground level beside tunnel as defined in Chapter 3

W = self-weight of tunnel

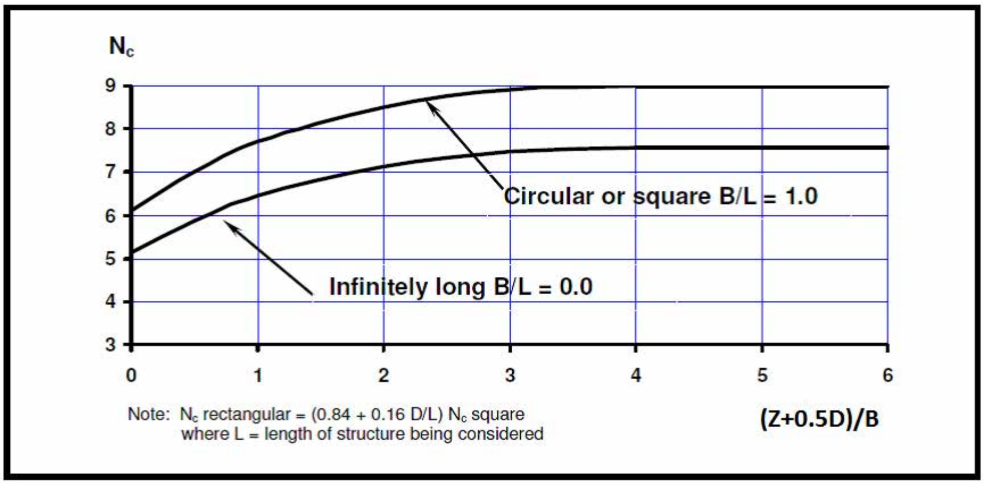
D = external diameter of tunnel

s_u = average c_u along a-a'

Note: For non-circular tunnels, the heave check shall be derived from above principle

7.5.4.3 The bearing capacity factor in the equation outlined in Figure 7.2 shall be based on Figure 7.3.

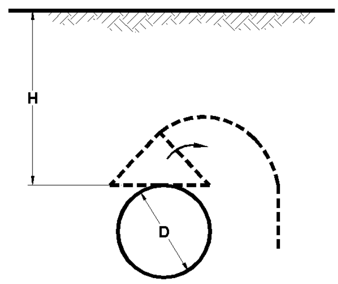
Figure 7.3 – Bearing capacity factor



7.5.4.4 In checking for uplift and heave, the self-weight of the tunnel shall include the weight of the lining only and it shall be divided by a model factor $\gamma_{S;d}$ of 1.05, when it is a stabilizing action.

7.5.4.5 For tunnels in clay, additional check for heave shall be carried out according to the following method (Figure 7.4). The resistance shall be divided by a model factor $\gamma_{R;d}$ of 1.5 for this additional check. The self-weight of soil type Estuarine (E), shall be multiplied with a model factor $\gamma_{S;d}$ of 1.2, when it is a destabilizing action.

Figure 7.4 – Additional possible heave verification



UPL limit state: $\gamma_E\, E_{dst;k} - \gamma_E\, E_{stb;k} \le R_k/\gamma_R$

Destabilizing action, $E_{dst;k} = \gamma_{bulk}\, {}^{\pi D^2}/_{4}$;

Stabilizing action, $E_{stb;k} = W$

Resistance, $R_k = D\, N_c\, c_u$

Where, γ_E = partial factor for the effect of an action

γ_R = partial factor for a resistance

W = self-weight of tunnel

D = external diameter of tunnel

by analogy with foundations:

N_c = 8.25 (after Meyerhoff[12])

γ_{bulk} shall be determined as the average value obtained from the site investigation occurring at the tunnel horizon, i.e. over the range of depth from tunnel crown to invert.

c_u shall be the design value corresponding to the depth of the tunnel axis.

Note: For non-circular tunnels, the heave check shall be derived from above principle

7.5.5 Longitudinal Stiffness

7.5.5.1 The design shall take into consideration that the modulus of subgrade reaction of soft clays that may be encountered in the invert of the tunnel may be so low that excessive deflections of the lining will occur under train or road vehicle loadings (as appropriate). The lining shall be designed to have adequate longitudinal stiffness to distribute the concentrated axle loading.

7.5.5.2 Detailed analysis of this problem shall be used to demonstrate and ensure that the following slope and deflection criteria in the longitudinal direction are not exceeded under train loading: -

a) Maximum slope of 1(V) to 2000 (H); and

b) Maximum total deflection of 3 mm.

Similar criteria shall be established for the design of road tunnels and submitted to the Engineer for acceptance.

7.5.5.3 The above analysis shall account for the cyclic nature of the loading. The stress levels shall be established to ensure that the fatigue life of the structure exceeds the specified design life.

7.5.5.4 The design of tunnels shall take into consideration of possible long-term movements due to all future construction in close proximity to the tunnels shown on the Authority's Drawings. Ongoing ground consolidation shall also be taken into consideration.

7.5.5.5 An analysis of long term movements of the tunnel shall be carried out to ensure that the tolerances and adjustment limits of the track are not exceeded;

a) Maximum allowable change in gradient: (Refer to the Particular Specifications and liaise with the trackwork designer to establish); and

b) Maximum adjustment to track: (Refer to the Particular Specifications and liaise with the trackwork designer to establish the limit of movement incorporated in the trackwork design, and shall ensure that this limit is not exceeded).

Similar criteria shall be established for the design of road tunnels and submitted to the Engineer for acceptance.

7.5.5.6 Special attention shall be paid;

a) The differential settlement and long term watertightness at the junction between bored/mined tunnel and cut & cover structure; and

b) At abrupt changes of ground conditions.

7.5.6 Deflections

The maximum deflection under any load combination shall not exceed 25mm on radius. The degree of flexibility of the lining shall be designed to ensure that this limit is not exceeded.

7.5.7 Additional Distortion

7.5.7.1 In addition to the deflections caused by the ground load, loading due to known developments identified in the documents and surcharge as appropriate, the lining shall be designed to accommodate an additional distortion of +/- 15mm on diameter to allow for future development.

7.5.7.2 This shall be analysed by reducing the horizontal / vertical ground load to produce an additional +/- 15mm distortion on diameter. Alternative ways of analysing the effects of additional distortion may be proposed for the acceptance of the Engineer.

7.5.7.3 The long-term concrete creep may be considered in the selection of the concrete Young's modulus. The moment of inertia of the segments may be reduced based on the recommendation of Muir Wood(1). The number of segments in a ring shall exclude any key less than half a standard segment in length.

7.6 PERMANENT TUNNEL LININGS REQUIREMENTS

7.6.1 Cast In-situ & Segmental Permanent Tunnel Linings

7.6.1.1 The analysis of the stresses induced in the final lining shall take account of the following: -

a) Short and long term horizontal and vertical ground loads in various critical combination of actions;
b) Method and Sequence of construction;
c) Proximity of other tunnels and structures;
d) Distortion of the cavity; and
e) Ground water loading.

7.6.1.2 For cast in-situ tunnel linings, the analysis shall include a 'wished in place' load case and a load case where only water pressure on the lining is considered.

7.6.1.3 The design of the final lining shall ignore any possible contribution to support of the imposed loads by the temporary support system.

7.6.1.4 In the evaluation of the design rock loads for permanent tunnel lining in rock, due account shall be taken of the proposed method of construction together with all temporary and permanent loads.

7.6.1.5 When the tunnel is in rock, analytical and/or finite element method shall be used for obtaining the ground loading acting on the permanent tunnel lining.

7.6.1.6 The design method shall take into account the long term loadings likely to be transferred from the rock to the permanent tunnel linings. All permanent tunnel linings shall be watertight and therefore designed for full hydrostatic loading from ground level and flood level.

7.6.1.7 Reinforced concrete (RC) linings shall be designed as short columns in accordance with SS EN 1992. The following requirements shall be met:

a) At the Serviceability Limit State, the calculated maximum crack-width shall not exceed 0.3 mm;

b) In respect of containment of compression reinforcement, the requirements of SS EN 1992 Clause 9.5.3 for columns shall apply.

7.6.1.8 In addition, transverse reinforcement shall be provided at a maximum spacing of 12 times the size of the smallest longitudinal bar.

7.6.1.9 Steel-fibre-reinforced concrete (SFRC) linings shall be designed in accordance with SS EN 1992, and shall remain uncracked for all combination of actions at Serviceability Limit State.

7.6.1.10 Drilled-in blind holes shall be positioned and detailed where required for the fixing of brackets, equipment and other fixtures.

7.6.2 Additional Requirements for Segmental Linings

7.6.2.1 General

7.6.2.1.1 Permanent segmental bored tunnel linings shall be in precast concrete with concrete strength class at least C50/60 with silica fume.

7.6.2.1.2 The design shall take into account, inter alia, the following: -

a) The width of segment shall suit the method of construction and shall not be so large that part-shoving of the shield becomes a necessity;

b) The width of segment shall be consistent with the capacity of the circumferential bolting arrangements to withstand any shear forces induced in linings built with staggered joints;

c) The stiffness of the lining shall comply with the deflection limit;

d) The length of segment shall be chosen with regard to bending stresses during handling and erection and the long-term stresses due to deflection and thrust;

e) Stresses induced by manufacturing and building tolerances including birdsmouthing of radial joints;

f) Stresses induced by handling, erection and jacking forward of the tunnelling shield;

g) Stresses induced by the tolerances on plane of the lining, steps between the edges of segments and the eccentricity between the shield jack and the segmental lining;

h) Stresses induced by grouting;

i) All metal fixings/reinforcement/inserts shall be detailed such that no electrical continuity will exist across the circumferential joints; and

j) A minimum nominal cover of 40 mm to all reinforcing bars shall be provided, except only at bolt holes where reduced cover of 25mm is acceptable provided a plastic sheath is cast in.

7.6.2.1.3 An arrangement of staggered radial joints shall be adopted. The induced shear across the circumferential flanges, in particular the shear on the circumferential joint connections, shall be considered in design.

7.6.2.1.4 The design shall allow for eccentricities of thrust between adjacent segments arising from the following factors: -

a) Building errors such as: lack of circularity, steps between segments and out of plane of circumferential joint. (Refer to M&W Specification for allowable tolerances)

b) Casting inaccuracies (Refer to M&W Specification for allowable tolerances)

c) Rotation under load: The rotation produced at each joint between segments as the lining deflects under load shall be calculated. The resultant eccentricity in the point of contact between segments from all these factors shall be calculated from the joint geometry.

7.6.2.1.5 The same design considerations shall apply in analysing the additional distortion.

7.6.2.1.6 The stresses in the segments arising from manufacturing, handling, erection and jacking forward of the tunnelling shield shall be considered in the analysis and suitable tolerances on these parameters to ensure that the segments remain uncracked.

7.6.2.1.7 For staggered radial joints, no increase in lining flexibility will be permitted in the design. However, when analysing for additional distortion the moment of inertia of the segments may be reduced based on the recommendation by Muir Wood (1). The number of segments in a ring shall exclude any key less than half a standard segment in length.

7.6.2.2 Bursting Stresses

7.6.2.2.1 For RC linings, bursting stresses shall be designed to be carried in full by reinforcement.

7.6.2.2.2 The bursting forces shall be calculated in accordance with SS EN 1992.

7.6.2.2.3 Where hot rolled high yield reinforcement is used to resist bursting the maximum allowable stress shall be 300N/mm^2 in Serviceability Limit State. For this purpose, the “tendon jacking force” shall be the maximum hoop load in the lining

7.6.2.2.4 For SFRC linings, the peak tensile bursting stress at ULS shall be less than the design tensile splitting strength of the SFRC material. The design tensile splitting strength shall be verified through laboratory testing to the satisfaction of the Engineer.

7.6.2.3 Bearing Stresses

For both RC and SFRC linings, the permissible concrete bearing stress at the Ultimate Limit State shall be calculated based on method given in SS EN 1992 Clause 6.7 for local crushing of concrete under partially loaded areas.

7.7 TEMPORARY TUNNEL LININGS REQUIREMENTS

7.7.1 The design of the lining in soft ground shall satisfy the combination of actions specified in section 7.4.1 and distortional loads given in section 7.4.2. For lining design in rock, the design loads shall be determined based on the proposed method of construction.

7.7.2 However, no allowance need to be made for additional distortion due to future developments unless there are specific requirements in the Particular Specification for any developments which will affect the tunnel prior to casting of the permanent lining. Any known development that could affect the tunnel shall be taken into consideration.

7.7.3 The temporary lining shall be designed such that: -

a) it will remain functional until the permanent lining has achieved full design strength

b) there is no loss of ground into the tunnel, whether by squeezing, erosion, loss of fines or any other cause

c) there is no damage to buildings, structures or utilities due to loss of ground, excessive deflection or consolidation settlements

d) shall be of adequate internal diameter to ensure that it does not infringe on the space required for the permanent lining taking into account of all construction, casting and build tolerances and of the lining deflection under load

7.7.4 Cracking of the temporary lining under construction loads is acceptable provided that there is no risk of loss of ground into the tunnel or of seepage causing consolidation settlements which could adversely affect buildings, structures or utilities.

7.8 SPRAYED CONCRETE LININGS (SCL)

7.8.1 SCL should not be carried out where the stand-up time for the excavation is inadequate. As a guide stand-up time should be more than 90 minutes for a face advance of 1 metre. Face stability assessments shall be conducted for representative sections along the length of the tunnel.

7.8.2 The design and construction methodology shall address the following: -

a) length of advance;
b) whether advance should be partial face or full face;
c) inclination of face;
d) speed of ring closure;
e) face support;
f) whether temporary invert is required; and
g) adjacent activities, such as excavations and ground treatment.

7.8.3 The design shall take account of the following: -

a) the ground conditions including:
   (i) soil stratigraphy;
   (ii) groundwater regime;
   (iii) soil strength, stiffness and small-strain characteristics;
   (iv) swelling and creep characteristics;
   (v) Poisson's ratio; and
   (vi) the bearing capacity of the ground at the foot of an unclosed arch and the possible need for enhancement such as 'elephant's feet'.

b) material properties including:
   (i) development of strength;
   (ii) stiffness (modulus) appropriate to the age of the concrete and the excavation stage; and
   (iii) creep and shrinkage especially in first two weeks after placing.

c) interaction between the ground and lining including non-linear and time dependent behaviour.

d) speed of loading, both horizontally and vertically including:
   (i) impact of any adjacent construction or ground treatment; and
   (ii) water pressure relief (or lack of relief).

7.8.4 In soft ground, full circle ribs shall be used at spacing equal to the length of advance.

7.8.5 The design shall be reviewed and modified during and after construction as a result of comprehensive monitoring of the following: -

a) the behaviour of the ground at the tunnel face in comparison with the design assumptions;

b) surface settlements;

c) lining deformations; and

d) measurements of ground loadings and displacements.

7.8.6 Appropriate contingency plans shall be prepared and implemented to modify the design and construction should the behaviour of the ground or the lining be shown by the monitoring to be exceeding the prediction limits.

7.9 SUPPORT SYSTEM IN ROCK

7.9.1 When the depth of rock cover is at least one time of tunnel diameter, the design of the support system using in rock shall be in accordance with the following recognized methods: -

a) Q-system for rock mass classification by Barton et al(5) combined with the latest updates published by Norwegian Geotechnical Institute (NGI)(6).

b) Hoek-Brown Failure Criterion by Hoek and Brown(7) combined with its latest updates(8) (9).

c) Rock Load Classification System by Terzaghi(10) and a graphical analysis of the steel arch rib after Proctor and White(713)

d) Finite element model.

7.9.2 The following key aspects shall be taken into consideration in the design:

a) axial and bending stresses in the steel arch ribs induced by the rock loads;

b) lateral stability and bracing of the steel arch ribs;

c) the method of forming the steel arch ribs and the resultant properties of the steel;

d) amount of preload to be applied to steel arch ribs and method of supplying this load;

e) method of blocking and spacing of blocking points;

f) bearing capacity of the rock at blocking points and, in the case of non-circular cross section, under the footplates;

g) the stand-up time of the unsupported part of the excavation;

h) the method of lagging between ribs to prevent ravelling and/or softening of the ground;

i) the ground water regime and permeability of the ground;

j) type, length and spacing of rock bolts;

k) design working load and test results of rock bolts;

l) type, length and spacing of forepoling;

m) method of grouting for forepoling;

n) thickness, layer of application and cover of shotcrete;

o) properties and test results of shotcrete; and

p) wedge stability analysis.

7.9.3 The steel arch ribs design shall be in accordance with SS EN 1993.

7.9.4 When the tunnel is in mixed face condition of soil and rock or the depth of rock cover is less than one tunnel diameter, the requirements in both Clause 7.8 and 7.9 shall be taken into account and the design shall be based on the most onerous analysis results from both clauses.

7.10 DETAILING

7.10.1 Fixings

7.10.1.1 Every available location on each tunnel segment shall be marked to indicate where drilling is permitted for the fixing of cable brackets and other attachments.

7.10.1.2 The marking shall take the form of indented areas, dimples or the like on the surface of the segment. The indentations shall not exceed 6mm in depth.

7.10.1.3 For the purpose of defining the location of the demarcated areas, it shall be assumed that holes to suit diameter 16mm expanding bolts or sockets will be used for the fixings. The clear distance between the sides of the hole and any reinforcement shall not be less than 40mm after making full allowance for inaccuracies in the position of the reinforcement.

7.10.2 Taper Rings

The Contractor shall design suitable taper ring linings in order to negotiate the alignment curvature and to correct for line and level during construction consistent with attaining the required degree of watertightness of the tunnels.

7.10.3 Bolt Pockets

Bolt pockets in which water may possibly accumulate shall be filled with strength class C25/30 concrete.

7.10.4 Grout Holes

7.10.4.1 Grout holes shall be provided in each segments and shall terminate short of the outer surface of the segment.

7.10.4.2 Where surface access is not available to carry out ground improvement at cross passage or sump locations and grouting for the purpose of ground improvement has to be done from within the tunnels, additional grout holes (with localized strengthening) with non-return valves shall be designed and provided for in the segments, subject to the Engineer's approval.

7.10.4.3 Foam Strip

Foam strip shall be provided along radial joints of segments, next to the extrados, to prevent tail void grout from by-passing the shield tail skin seal into the tunnel boring machine. The foam strip shall be compressible so that it does not exert additional load onto tunnel segments under permanent loading conditions.

7.11 WATERPROOFING

7.11.1 General

7.11.1.1 In order to minimize surface settlements, the specified degree of water tightness shall be achieved in accordance with the requirements specified in M&W Chapter 16.

7.11.1.2 Notwithstanding the limits on ground water seepage rates given in M&W Chapter 16, the Contractor shall ensure that no loss of ground occurs through any part of the completed structure.

7.11.1.3 For bored tunnels the design shall make provision to prevent the build-up of water pressure beneath the first stage concrete or beneath the track slab concrete. To achieve this, drainage paths shall be created at each joint in the tunnel lining and any seepage directed to the drainage system.

7.11.2 Waterproofing for Segmental Linings

The waterproofing method throughout the segmental tunnel linings shall include the use of the following: -

a) A co-extruded single composite gasket consisting of an elastomeric carrier and hydrophilic facing material;

b) The elastomeric carrier shall be an EPDM (Ethylene Propylene Diene Monomer) formulated to provide good retention of elasticity and low stress relaxation properties;

c) The outer (convex) surface of all segments that contain steel rebar, together with all side faces, gasket recesses, caulking grooves and insides of bolt holes and grout holes shall be painted with a solvent free (or water based) emulsion epoxy coating;

d) The composite gasket shall not be in front of the bolt hole; and

e) Provision for caulking around all edges of segments.

7.11.3 Waterproofing for Cast In-situ Linings

7.11.3.1 A waterproofing membrane system consisting of a protective geotextile fleece fastened to shotcrete surface and a compartmentalised and fully welded membrane, shall be provided between the temporary lining and permanent cast in-situ concrete lining including at cross passageways and sumps. The watertightness of the membrane system, strength class of concrete, thickness of lining, method of placement, treatment of construction joints and arrangements for re-injectable leak sealing system shall be chosen such that the specified watertightness can be achieved.

7.11.3.2 As alternative to the waterproofing membrane system, the use of spray applied waterproofing may be allowed for cross passageways and sumps, subject to Engineer’s approval. At such locations, an integral waterproofing system comprising of an approved sprayed waterproofing membrane and the permanent cast in-situ concrete lining with either an approved waterproofing admixture or silica fume shall be provided.

7.11.3.3 At the construction joint of permanent cast in-situ concrete lining and at the interface between cut-and-cover tunnels, cross passageway and sumps with the bored tunnel lining, the arrangement of re-injectable grout tubes and hydrophilic strips together with longitudinal and transverse waterstops shall be designed and implemented to ensure long-term watertightness of underground infrastructures.

7.12 CROSS PASSAGEWAYS BETWEEN RUNNING TUNNELS

7.12.1 Location

7.12.1.1 For railways, cross passageways shall be located in accordance with the requirements of the relevant codes issued by the Singapore Civil Defence Force. For roads, cross passageways between tunnels shall be located in accordance with NFPA 502 and BD 78/99.

7.12.1.2 Track crossovers in railway tunnels and accesses for emergency vehicles in road tunnels shall not be considered as cross-passages.

7.12.2 Dimensions and Layout

7.12.2.1 Throughout the length of the cross passageway both the minimum clear headroom and the minimum clear width shall comply with the requirements of the Building Control Regulations. The door shall comply with the relevant codes from the Singapore Civil Defence Force.

7.12.2.2 For railways the level on the cross passageway floor shall be determined, on a case by case basis, in relation to the cant of the track. Generally, the cross passageway level shall be above the adjoining trackbed level as determined by the trackwork designer. For roads, the level shall be proposed by the designer for the acceptance of the Engineer.

7.12.2.3 The cross passageway floor shall drain into the running tunnel drainage system, unless the cross passageway occurs at the low point of the alignment in which case a drainage sump may be located within the cross passageway.

7.12.2.4 Openings in cross passageways shall be protected with a fire door assembly having a fire resistance of not less than 2 hours with a self-closing device. The dimension of the door shall comply with the relevant codes from the Singapore Civil Defence Force. The door shall open and swing in the direction of the cross passageway.

The fire rated door shall be designed to resist actions in accordance with Chapter 3.

7.12.3 Design Requirements

7.12.3.1 The cross passageway tunnel lining shall be designed generally in accordance with the requirements of Clause 7.5 above as appropriate with the following exceptions:

a) The maximum allowable deflection on radius shall be 15 mm and

b) Taper rings will not be required.

7.12.3.2 The junctions with the running tunnels shall be steel framed and encased with in-situ concrete or framed with reinforced concrete. The junctions shall be designed to fully support the running tunnel linings at the openings together with the ground and ground water loads on the junction itself.

7.12.3.3 Where openings are to be formed in running tunnels having segmental concrete linings the Contractor shall provide temporary internal supports to the running tunnel lining. These supports shall adequately restrain the lining such that on completion of the cross passageway and removal of the temporary supports the total deflection of the lining does not exceed the requirements of Clause 7.5.6.

7.13 SUMPS IN RUNNING TUNNELS

7.13.1 Refer to Chapter 11 for design requirements for pump sumps in running tunnels.

7.13.2 Where possible, the sump shall be located within the cross passageway. The sump shall not be located at mixed face geology unless otherwise accepted by the Engineer.

7.14 EMERGENCY ESCAPE SHAFTS

7.14.1 Location

For railways, emergency escape shafts shall be located in accordance with the requirements of the relevant codes issued by the Singapore Civil Defence Force. For roads, emergency escape shafts shall be located in accordance with NFPA 502 and BD 78/99.

7.14.2 Dimensions and Layout

7.14.2.1 The floor of the shaft shall be level with the walkway level in the tunnel.

7.14.2.2 For railways the layout of the shaft shall conform to the relevant codes issued by the Singapore Civil Defence Force and Architectural Design Criteria (ADC) by Land Transport Authority.

7.14.3 Design Requirements

7.14.3.1 Escape shaft shall be designed generally in accordance with Chapter 8.

7.14.3.2 The interface with the running tunnels shall be designed to support the ground and ground water loads at the junction, openings to the running tunnels and the shaft.

7.14.3.3 While the openings into the running tunnel and the shaft are being formed, temporary internal supports to the running tunnel lining and shaft lining shall be provided. These supports shall adequately restrain the linings such that on completion of the junction and removal of the temporary supports the total deflections of the linings do not exceed the requirements of Clause 7.5.6.

7.15 TUNNEL WALKWAY

7.15.1 Arrangement

The location and width of the tunnel walkway are shown on Authority's drawings.

7.15.2 Details of Walkway

7.15.2.1 The walkway shall fall 15mm towards the track or road.

7.15.2.2 The walkway shall be ramped down to cross-passage floor level at each cross passageway and in railway tunnels shall be ramped down to rail level at switch and crossing areas. In road tunnels the walkway shall be ramped down to road level at vehicle crossover areas.

7.16 FIRST STAGE CONCRETE IN RAILWAY TUNNELS

7.16.1 The first stage concrete is defined as the concrete between the tunnel lining and the track concrete placed by the trackwork contractor. The first stage concrete, or concrete on which the second stage is cast, should have an exposed aggregate finish.

7.16.2 The minimum concrete strength class shall be C30/37.

References

1. Muir Wood AM. The circular tunnel in elastic ground. Geotechnique 25 No.1, 1975.

2. Curtis DJ. Discussion on 3 above. Geotechnique 26 No.1, 1976.

3. Duddeck H and Erdmann J. Structural design models for tunnels. Tunnelling 1982, pp 83-91

4. Peck RB. Deep excavation and tunnelling in soft ground. State of the Art Report. 7th International Conference on Soil Mechanics and Foundation Engineering, Mexico City 1969.

5. Barton N, Lien R and Lunde J. Engineering classification of rock masses for the design of tunnel supports. Rock Mechanics Vol.6 No.4, 1974.

6. Norwegian Geotechnical Institute Using the Q-system – Rock mass classification and support design. Handbook published by NGI, 2013.

7. Hoek E and Brown ET. Empirical strength criterion for rock masses. J. Geotechnical Engineering Division ASCE, 1980.

8. Hoek E, Carranza-Torres CT and Corkum B. Hoek-Brown failure criterion – 2002 edition. Proceedings of the fifth North American rock mechanics symposium, 2002

9. Hoek E, and Diederichs MS. Empirical estimation of rock mass modulus. International Journal of Rock Mechanics & Mining Sciences 43 (2006) 203–215

10. Terzaghi K. Rock tunnelling with steel supports, Section 1. Commercial Shearing and Stamping Co. 1946.

11. Bjerrum L. and Eide O. Stability of strutted excavations in clay. Geotechnique 6, 1956.

12. Meyerhoff GG. The ultimate bearing capacity of foundations. Geotechnique Vol.2 No.41, 1951.

13. Proctor and White Rock tunnelling with steel supports. Commercial Shearing and Stamping Co. 1946.

CHAPTER 8

UNDERGROUND STRUCTURES

8.1 SCOPE

8.1.1 This chapter covers the design requirements for depressed or underground structures constructed by cut and cover methods.

8.2 STANDARDS AND CODES OF PRACTICE

8.2.1 The design shall comply with SS EN 1990, SS EN 1991, SS EN 1992, SS EN 1997, their Singapore National Annexes, other relevant codes and the additional requirements herein.

8.3 ACTIONS

8.3.1 Partial Factors for Earth and Water Pressure

8.3.1.1 The partial factors for earth and water pressure for the structures shall be according to the partial safety factors tables as shown in Figures 8.1 to 8.4.

8.3.1.2 Refer to Chapter 3 for the definition of Design Ground Water Level at permanent condition and ground water level at accidental condition.

8.3.1.3 For the groundwater pressure, the partial factors in Table 8.1 shall be used for the ultimate limit state design of underground structures.

Table 8.1 - Partial Factors for Groundwater Pressure in Structural Design

Groundwater Condition	Partial Factor
Permanent Condition	1.35
Accidental Condition	1.0

8.3.2 Ground Actions

8.3.2.1 Refer to Chapter 3 for additional requirement due to earth pressures that shall be considered in design. The design parameters for earth pressures are given in Chapter 5.

8.3.3 Combination of Actions

8.3.3.1 General

The various possible combinations of permanent and variable actions and/or known future development shall be considered in deriving the most onerous combinations of actions for underground structures. The envelopes of the stress resultants at both ultimate and serviceability limit states shall be determined from the critical combinations listed in Table 8.2.

Table 8.2 – Critical Combination of Actions for Design of Underground Structures

Combination of Actions	**Description**
Combination 1	Maximum vertical, maximum horizontal, including end of construction stage (Figure 8.1)
Combination 2	Maximum vertical, minimum horizontal (Figure 8.2)
Combination 3	Minimum vertical, maximum horizontal (Figure 8.3)
Combination 4	Unbalanced forces (Figure 8.4)
Any other combination of actions that shall be determined to be more onerous.	

8.3.3.2 The reduction factor ψ under Category C – Congregation Area in Table NA.A1.1 of NA to SS EN 1990 shall be used for representative values of variable actions. The values of the reduction factor are listed in Table 8.3.

Table 8.3 Reduction Factor for Variable Actions

Reduction Factor	**Value**
ψ_0	0.7
ψ_1	0.7
ψ_2	0.6

8.3.3.3 Underground structures shall be designed for unbalanced actions and differential settlements due to the future development(s) identified in the Particular Specification.

8.3.3.4 In applying the actions in Combination 4, the base of the structure shall be assumed to be rigidly restrained horizontally and to be restrained vertically to correspond to the actual foundation conditions. It may be assumed that the unbalanced action is also restrained by a reactive pressure generated by the mobilisation of the soil stiffness on the reacting side. For closed box structures or open "U" structures with cantilever walls, if under serviceability limit state, the differential settlement across the width of the structure exceeds 1:1000, the differential action shall be reduced by increasing the lower earth pressure until the differential settlement is less than or equal 1:1000. In addition, the horizontal movement of the top of the closed box shall not exceed 15mm. In either case, the out-of-balance action shall be combined with other vertical conditions that may co-exist. For the serviceability limit state of cracking under this combination, the calculated maximum crack width due to flexure on both external and internal faces of those members forming the hull of the underground structures shall not exceed 0.3mm.

8.3.3.5 When the horizontal differential movement of the structure under maximum unbalanced actions is less than 15mm, such information shall be stated in the drawings and provided to LTA Development and Building Control (DBC) Department for updating the allowable limits for movement and incorporation in the Code of Practice for Railway Protection.

8.3.3.6 Separate combination of actions shall be developed for the design of internal elements. Internal elements shall be designed for displacement compatibility with the hull elements under the combination of actions used. (For example, where internal columns are supported on base slabs which deflect relative to the side walls due to external soil or hydrostatic pressure, this should be allowed for in the design of the internal elements).

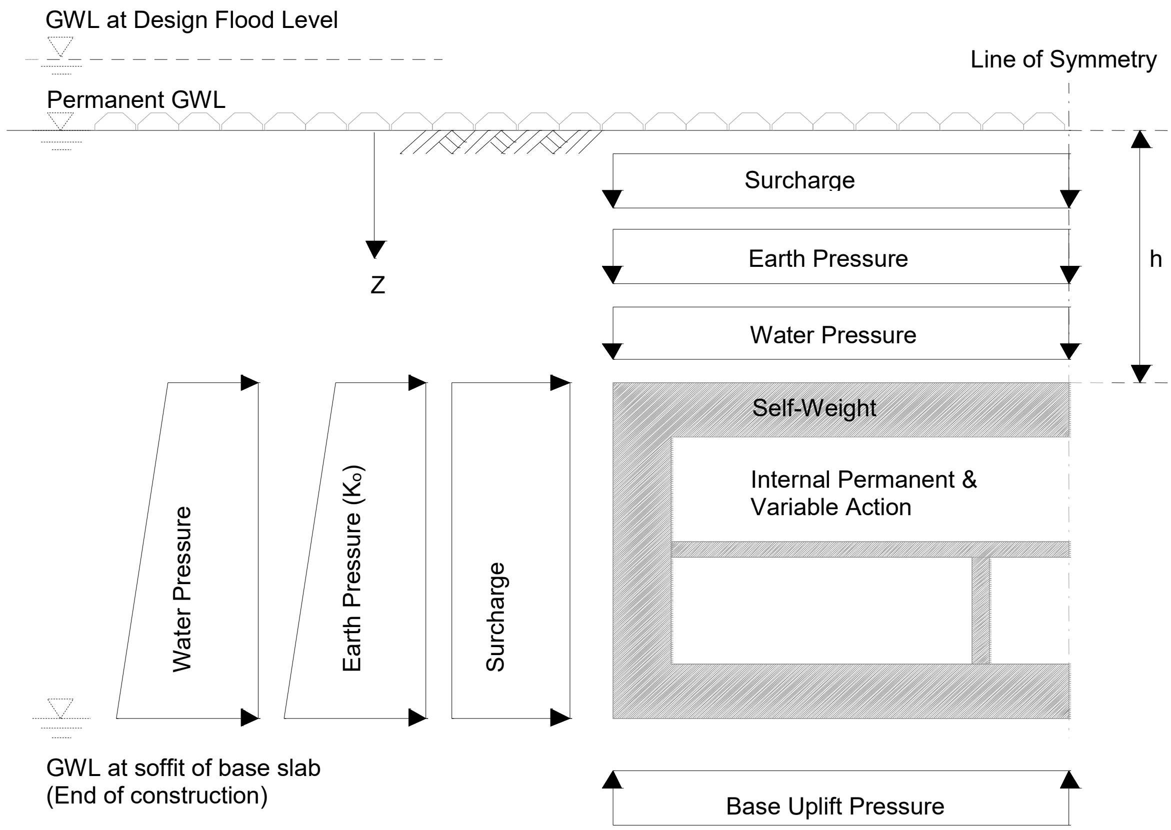
8.3.3.7 For designing of structures with diaphragm walls as part of permanent structures, the design forces on the walls should be obtained from the superposition (addition) of Case 1 and 2, and then envelope with Case 3, where

a) Case 1
Forces on the walls from geotechnical finite element analysis at the final stage of construction, taking into consideration the full construction sequence.

b) Case 2
Forces on the walls from structural finite element analysis of a wished-in-place model, which consider long term actions for the permanent stage of the structures. Only Combinations 1 and 3 need to be considered for the “wished-in-place” model. The lateral earth pressure adopted shall be the difference between at rest (K_o) earth pressure and the earth pressure at the last stage of construction from Case 1.

c) Case 3
Forces on the walls from structural finite element analysis of a wished-in-place model for all combination of actions using the appropriate lateral earth pressure as required in each of the combination of actions.

Figure 8.1 – Combination of Actions 1
(Maximum Vertical & Maximum Horizontal Action)



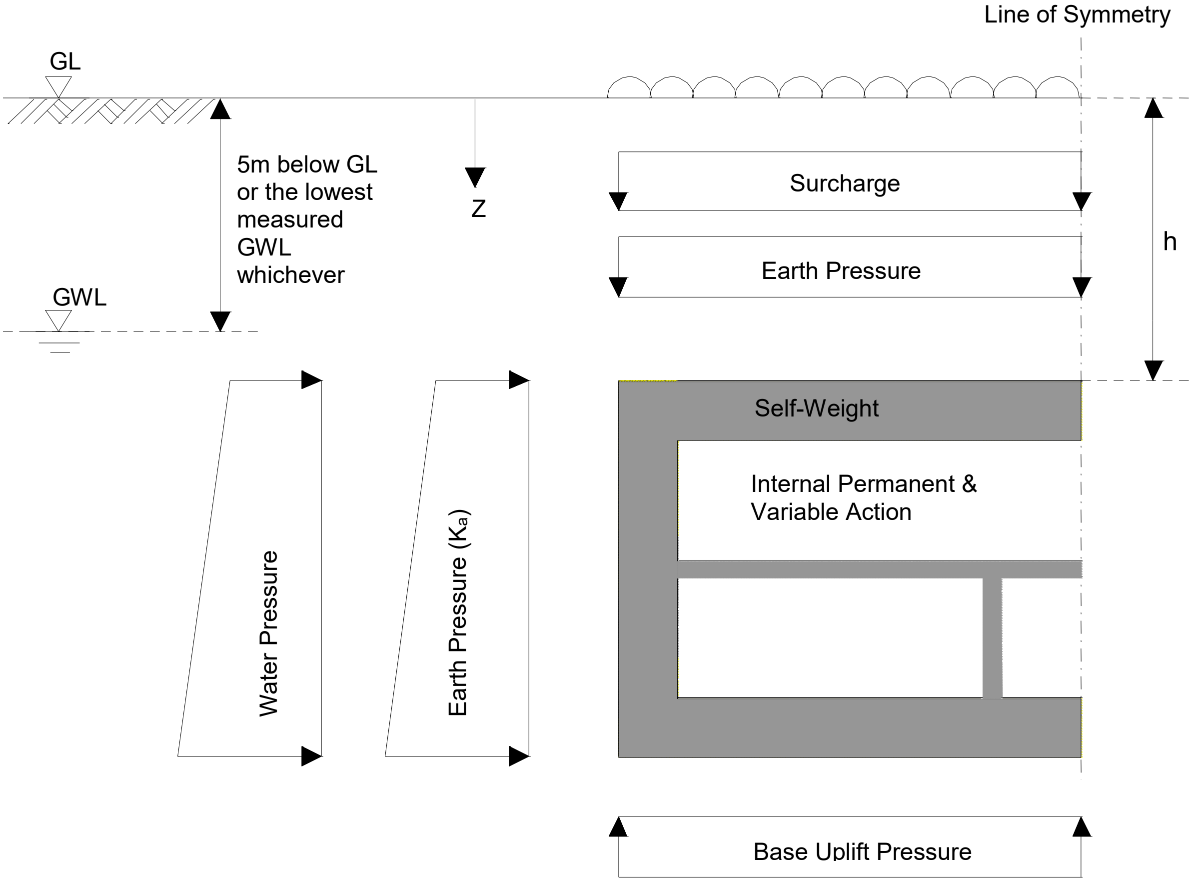
Notes:

1) *Water pressure shall vary according to ground water levels (GWL). This combination of actions shall consider the most onerous of all 3 specified GWL with different partial factors:*
   a) *GWL at GL in permanent condition*
   b) *GWL at design flood level (FL) in accidental condition*
   c) *GWL at soffit of base slab level at end of construction case (not applicable for top down construction)*
2) *When GWL is at FL, surcharge need not be considered.*

Partial Factor Table

Action		Vertical surcharge	Vertical Earth Pressure	Vertical water pressure		Horizontal surcharge	Horizontal earth pressure (K_o)	Horizontal water Pressure		Base uplift pressure		Self-Weight	Internal Permanent Action	Internal Variable Action
				Permanent GWL	Accidental GWL (at FL)			Permanent GWL	Accidental GWL (at FL)	Permanent GWL	Accidental GWL (at FL)			
Partial Factor	**ULS**	**1.5**	**1.35**	**1.35**	**1.0**	**1.5**	**1.35**	**1.35**	**1.0**	**1.35**	**1.0**	**1.35**	**1.35**	**1.5**
	SLS	**1.0**	**1.0**	**1.0**	**1.0**	**1.0**	**1.0**	**1.0**	**1.0**	**1.0**	**1.0**	**1.0**	**1.0**	**1.0**

Figure 8.2 – Combination of Actions 2
(Maximum Vertical & Minimum Horizontal Action)

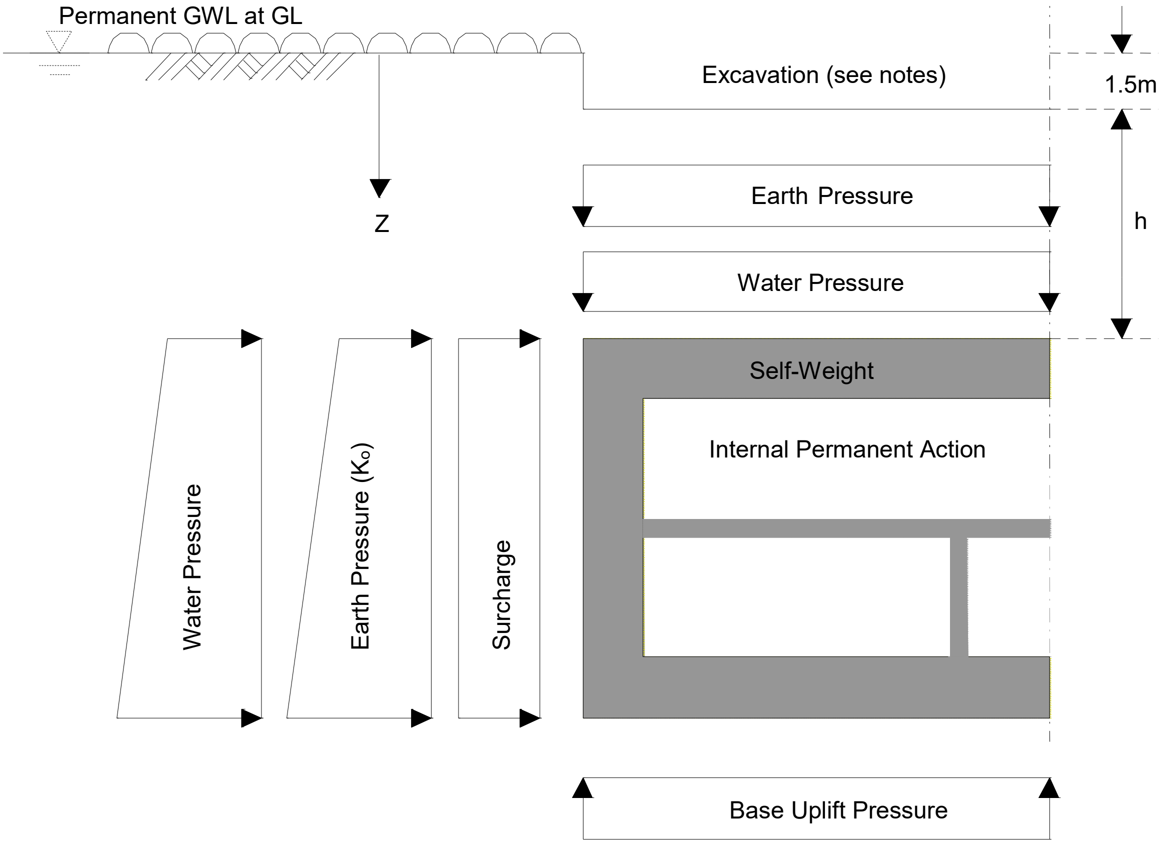


Notes:
1) *The active earth pressure (K_a) is used for minimum horizontal action.*

Partial Factor Table

Action		Vertical surcharge	Vertical Earth Pressure	Vertical water pressure	Horizontal earth pressure (K_a)	Horizontal water Pressure	Base uplift pressure	Self-Weight	Internal Permanent Action	Internal Variable Action
Partial Factor	**ULS**	**1.5**	**1.35**	**1.35**	**1.0**	**1.0**	**1.35**	**1.35**	**1.35**	**1.5**
	SLS	**1.0**	**1.0**	**1.0**	**1.0**	**1.0**	**1.0**	**1.0**	**1.0**	**1.0**

Figure 8.3 – Combination of Actions 3
(Minimum Vertical & Maximum Horizontal Action)

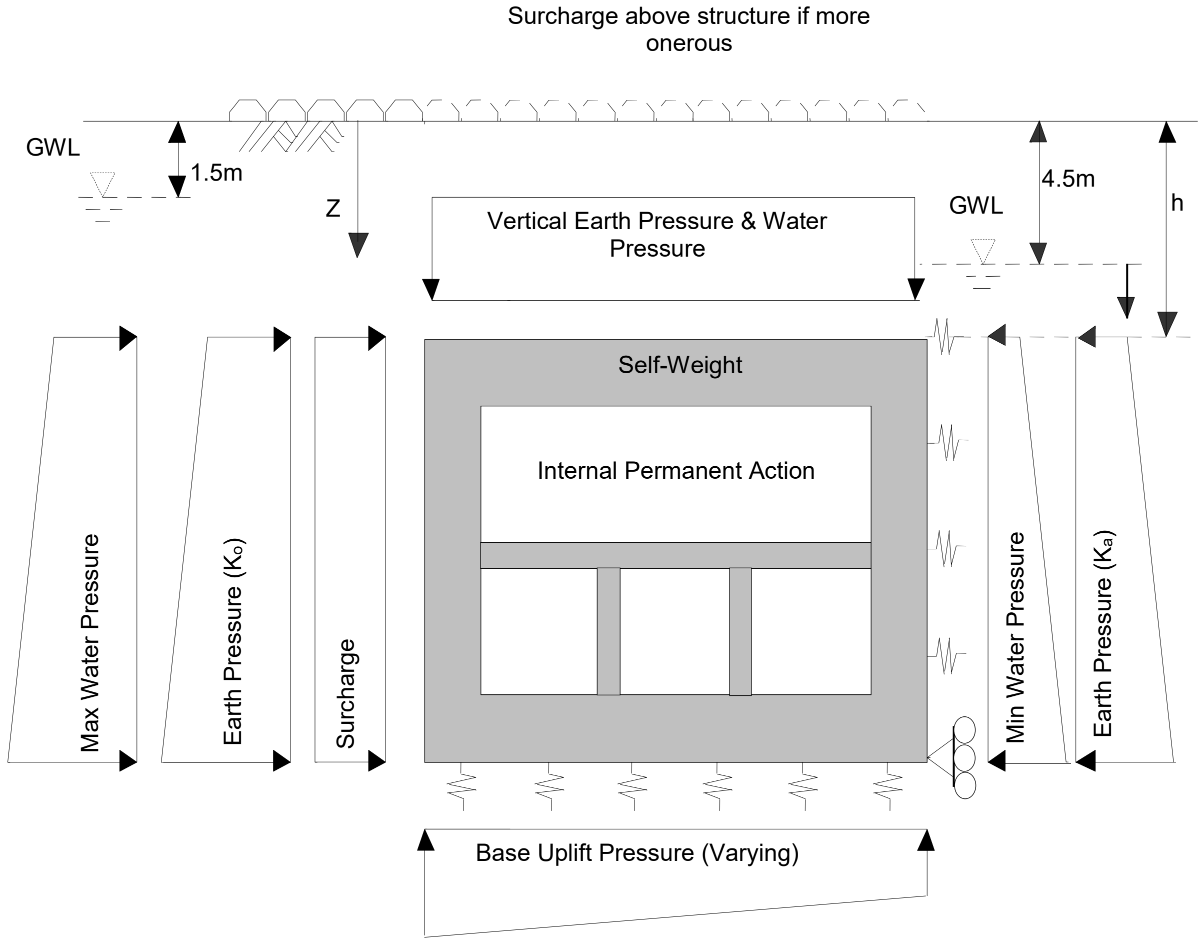


Notes:
1) *Excavation on top of roof slab of 1.5m is allowed. This thickness of overburden shall be ignored in consideration for minimum vertical action on the structure.*

Partial Factor Table

Action		**Vertical Earth Pressure**	**Vertical Water Pressure**	**Horizontal Surcharge**	**Horizontal Earth Pressure (K_o)**	**Horizontal Water Pressure**	**Base Uplift Pressure**	**Self-Weight**	**Internal Permanent Action**
Partial Factor	**ULS**	**1.0**	**1.0**	**1.5**	**1.35**	**1.35**	**1.0**	**1.35**	**1.35**
	SLS	**1.0**	**1.0**	**1.0**	**1.0**	**1.0**	**1.0**	**1.0**	**1.0**

Figure 8.4 – Combination of Actions 4 (Unbalanced Action)



Notes:
1) *The earth pressure shall be in accordance to ground level or the lowest excavation level as stated in the Particular Specification.*

Partial Factor Table

Action		Vertical Surcharge	Vertical Earth Pressure	Vertical Water pressure	Horizontal Surcharge	Horizontal Earth Pressure	Horizontal Water Pressure	Base Uplift Pressure	Self-Weight	Internal Permanent Action
Partial Factor	**ULS**	**1.5**	**1.35**	**1.35**	**1.5**	**1.35**	**1.35**	**1.35**	**1.35**	**1.35**
	SLS	**1.0**	**1.0**	**1.0**	**1.0**	**1.0**	**1.0**	**1.0**	**1.0**	**1.0**

8.4 DESIGN REQUIREMENTS

8.4.1 General

8.4.1.1 Reinforced concrete is preferred for the permanent works. Prestressed concrete structure not forming the external hull structure may be considered, subject to the Engineer's acceptance.

8.4.1.2 The primary design objective of underground structures is that they shall be made watertight by the provision of impermeable dense durable concrete. The structure shall be capable of withstanding aggressive soil and water conditions that may be present.

8.4.1.3 The design of cut and cover structure shall at least take into account of the following:

a) Geology;

b) Hydrogeology and strata permeabilities in the vicinity of the excavation;

c) Degree of lateral movement and settlement which would be expected. In this context the location of the works in relation to existing structures shall be considered;

d) Method and sequence of construction;

e) Construction tolerances including eccentricity and verticality;

f) Delivery and replacement of equipment;

g) Control over heave and instability of the base of the excavation, and long term settlement and heave;

h) Methods by which the completed structure shall be secured against flotation; and

i) Method for waterproofing of the completed structure

The relevant information shall be clearly specified on the drawings.

8.4.1.4 The underground structures shall be designed to be structurally continuous, no movement joint is allowed. The ground pressure distribution, long term ground/structure interaction, total and differential settlement shall be considered accordingly.

8.4.1.5 Capping beams shall be provided for contiguous bored pile and secant pile walls, or for diaphragm walls as a cantilevered earth retaining structure.

8.4.1.6 The design of temporary/permanent wall shall have sufficient cut-off to minimise the reduction in piezometric pressure in the adjacent soils. Dewatering shall avoid lowering of the water tables outside the Works.

8.4.1.7 All underground structures shall have a minimum ground cover of 2 metres, measured from top of waterproofing's protection layer to the lowest point on the finished ground level unless otherwise stated by the Engineer.

8.4.1.8 All openings to underground structures shall be designed to meet the Design Crest Level as specified in Public Utilities Board (PUB)'s Code of Practice on Surface Water Drainage.

8.4.2 Robustness

The underground structure shall be designed for accidental actions with adequate provision of ties to ensure robustness in accordance with SS EN 1992-1-1.

8.4.3 Settlement

The underground structure shall be designed for settlement including differential settlement according to Chapter 6.

8.4.4 Crack Width

8.4.4.1 Those members exposed to earth and/or ground water and forming the hull of underground structures shall be designed such that the calculated maximum crack width due to early thermal cracking or flexure and/or tension shall be limited to:

a) 0.25mm for external face when subject to persistent forces

b) 0.30mm for external face when subject to transient forces which are not "locked-in" during construction

c) 0.30mm for internal face

8.4.4.2 The crack width shall be calculated based on forces derived from Serviceability Limit State (SLS) quasi-permanent combinations (ψ_2=0.6) and:

- at 40mm from the outermost reinforcement; or
- at the cover required for durability ($C_{min,dur}$) based on SS 544 (excluding the deviation for construction tolerance Δc);

whichever is more onerous.

8.4.4.3 The calculated crack width need not be added to those caused by early-age thermal cracking and shrinkage.

8.4.4.4 The persistent compression force within the underground structure can be considered when calculating the crack width due to combined flexure-compression effect. This compression force shall not include any beneficial actions from completed or future developments supported by the underground structures, unless otherwise accepted by the Engineer.

8.4.5 Flotation

8.4.5.1 Underground structures shall be checked for the possibility of flotation at all stages of the construction and throughout the service life of the structure. In the permanent condition, ground water level shall be assumed to be at Design Flood Level. Any actions from developments or from any other structure that would be beneficial to stability against flotation shall not be considered in the flotation assessment.

8.4.5.2 The assessment of the general case shall be as shown in Figure 8.5. The partial factors under Uplift Limit State (UPL) in SS EN 1997-1 shall be used for flotation assessment.

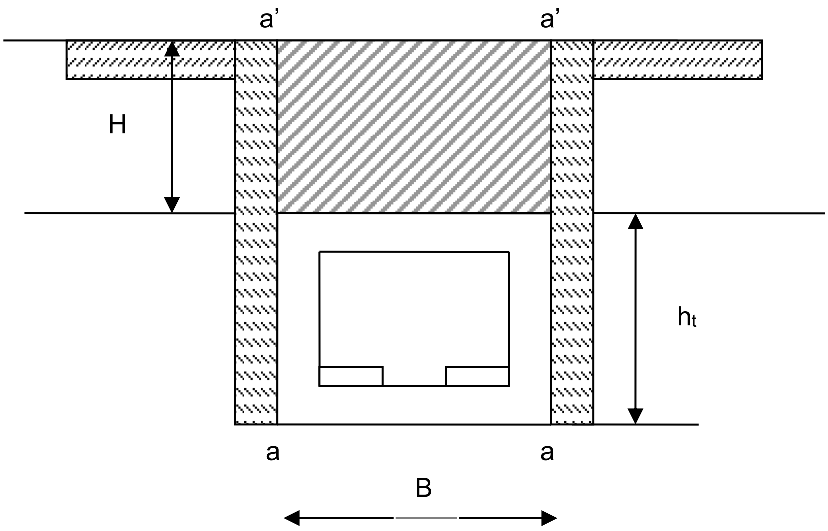
8.4.5.3 Shear Resistance of Soil

The partial factors for soil parameters under Uplift Limit State (UPL) in SS EN 1997-1 shall be used for flotation check. For cohesive soils, an adhesion factor shall be determined from suitable published data (e.g. M.J Tomlinson (2001). Foundation Design and Construction), and for cohesionless soils earth pressure coefficients taking into account the effects of the following as appropriate:

a) The shear strength of the backfill;
b) The method of placing of backfill material;
c) The temporary support system, either left in place or extracted;
d) Grouting;
e) The use of bentonite;
f) The depth below ground surface; and
g) The waterproofing system for the structure.

With respect to item (g) above, where the critical shear interface is along the waterproofing membrane no frictional resistance shall be used. No shear resistance shall be allowed within 2 metres of the ground surface.

Figure 8.5 – Flotation Assessment



Considering 1m run of the structure,

Uplift force, $E_d = \gamma_w \, h_t \, B \, \gamma_{G;dst}$

Where, γ_w = unit weight of water

h_t = height of structure

B = width of structure

Restraining force, $R_d = \gamma' \, B \, (H - 1.5) \times \gamma_{G;stb} + (h_t + H - 2)2S/\gamma_M + W \times \gamma_{G;stb}$

Where, γ' = submerged weight of backfill material

S = average shear resistance along a - a'

W = self-weight of structure

H = depth of backfill

B = width of structure

$\gamma_{G;dst}$ = partial factor for unfavourable permanent actions = 1.1 *Note 1

$\gamma_{G;stb}$ = partial factor for favourable permanent actions = 0.9 *Note 2

γ_M = partial factor for soil parameters

$R_d \geq E_d$

Note 1: Since the design water table is above the finished ground level, the effective weight of the backfill shall be based on the submerged density of the material. In the calculations, backfill within the top 1.5m of the ground surface shall be ignored.

Note 2: For railways, first stage concrete (if any) may be considered as self-weight of the structure. Weight of partition walls, floor finishes, road surfacing, false ceiling, equipment and other superimposed actions, etc. shall not be considered.

8.4.5.4 Measures to Counteract Flotation

Unless otherwise accepted by the Engineer, the vertical alignment of the tunnels should not be modified to counteract the flotation forces. The use of ground anchors as a permanent measure to counteract flotation forces shall not be permitted. Suitable measures to counteract flotation forces shall be incorporated in the design. The measure(s) chosen shall suit the particular conditions and method of construction and may include:

a) Toeing in of the base slab into the surrounding ground or fill. Where the base slab is toed into the surrounding ground or fill, the shear resistance may be obtained from the shear resistance of the ground or fill as appropriate. The shear resistance of the ground or fill above the toe shall be divided by the partial factor γ_M under Uplift Limit State (UPL). The value of the weight of ground above the toe shall be calculated as for the backfill material, unless mass concrete is used. Where toes are provided, the minimum toe projection shall be 0.5m.

b) The value of the weight of any additional thickness of concrete shall take into account the increased volume of water displaced. Increasing the self-weight of the structure by:
   (i) Thickening of structural members.
   (ii) Providing an extra thickness of concrete beneath the base slab tied into the structural base slab.
   (iii) Deepening diaphragm walls.

c) The provision of tension piles. The tension force in DFEs shall be calculated taking into consideration the stiffness of the DFEs and their associated structural frames.

8.4.6 Stability of Completed Structure

The stability of the completed structure against failure due to base heave under the structure shall be checked in accordance to SS EN 1997-1, Hydraulic Heave Limit State (HYD). The assessment of the general case shall be as shown in Figure 8.6.

Figure 8.6 – Stability Check of the Structure

The effects of actions:

$E_d = \gamma_{G;dsb} \times \gamma D + \gamma_Q \times q$

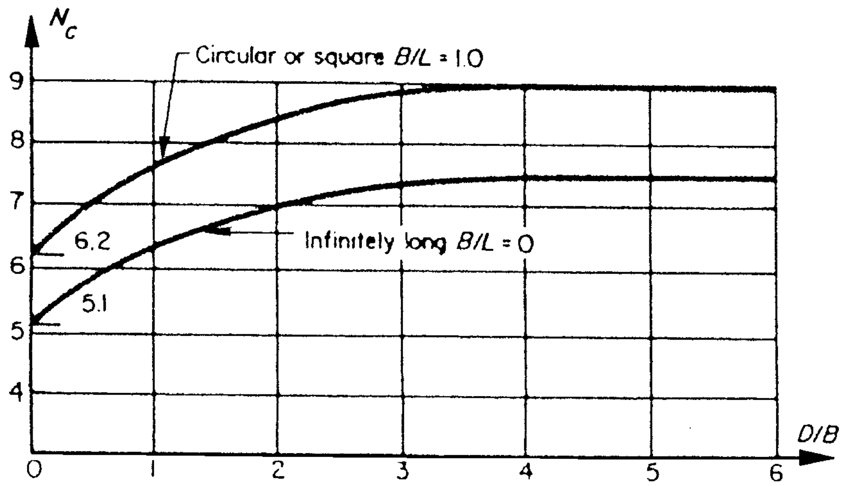
The resistance:

$R_d = N_c \times C_u / \gamma_M + \gamma_{G;stb} \times p$

Where,

C_u = shear strength of clay in zone of base of structure
D = depth to base of structure from ground level
Q = surcharge at ground level beside structure (Refer Chapter 3)
N_c = bearing capacity factor
P = resistance of completed structure to uplift expressed as a pressure at base level
γ = average bulk density of soil above level of base of structure
$\gamma_{G;dst}$ = partial factor for unfavourable permanent actions = 1.35
$\gamma_{G;stb}$ = partial factor for favourable permanent actions = 0.9
γ_Q = partial factor for unfavourable variable actions = 1.5
γ_M = partial factor for soil parameters

$R_d \geq E_d$



Note :

N_c rectangular = $(0.84 + 0.16\ B/L)\ N_c$ square

where B = width of the foundation, and
L = length of the foundation

8.4.7 Redistribution of Moments

Modification to Clause 5.5 of SS EN 1992-1-1 and NA to SS EN 1992-1-1.

8.4.7.1 Redistribution of moments is applicable only for structures designed to SS EN 1992 at the Ultimate Limit State (ULS). Clause 5.5 of SS EN 1992-1-1 and its associate National Annex apply except that the redistributed moment shall not exceed 10% of the elastic bending moment, i.e. the minimum redistribution ratio δ is 0.9.

8.4.7.2 Redistribution of stress resultants will not be permitted in allowing for locked-in stress resultants.

8.4.8 Design Moments at Supports

8.4.8.1 In analysis of continuous and rigid frame members, distances to the geometric centres of members shall be used for the determination of moments. Where members are integral with (i.e. monolithic width) their supports, the design support moment may be taken at the face of the support.

8.4.8.2 Where members are not designed integrally with their supports which offer no rotational restraint, the moments at supports shall be taken as the centre-line peak moments. This moment may be reduced based on Clause 5.3.2.2 (4) of SS EN 1992-1-1 and the reduction shall not exceed 10% of the centre-line peak moment.

8.4.9 Internal Facing of Diaphragm and Secant Pile Walls

8.4.9.1 An internal facing wall of in-situ reinforced concrete is mandatory for secant pile and similar walls that are incorporated into the permanent works. For diaphragm walls, an internal facing wall is not mandatory unless specified elsewhere.

8.4.9.2 Irrespective of whether or not composite action is assumed, the internal facing wall shall be designed for full hydrostatic pressures, with cast-in bent-out bars from the permanent external wall. The full thickness of the internal facing wall shall be of waterproofing concrete.

8.4.9.3 When composite action is assumed, interface shear reinforcement bars anchored into the remote faces of the two walls shall be provided to prevent interface separation and slippage under full hydrostatic pressures. The amount of reinforcement bars shall be the higher of that required to prevent separation or for hydrostatic pressure.

8.4.9.4 It shall be noted that both in-situ facing and cavity wall construction may be necessary in some circumstances stipulated in Chapter 11.

8.4.10 Connections between Bored Tunnels, Cut-and-Cover Tunnels and Station

8.4.10.1 Where bored tunnels are connected to cut-and-cover structures, the connection shall be designed so that completion of the joint is carried out by the contractor for the cut and cover structure.

8.4.10.2 Design of the joint between bored tunnels, cut-and-cover tunnels and/or stations shall consider the possibility of differential movement due to backfilling sequence, different overburden and foundation types. The construction sequence shall be clearly shown on the drawings.

8.4.10.3 Particular attention shall be paid to the waterproofing detail between bored tunnels and cut-and-cover structures, to ensure the watertightness of the joint.

8.5 DURABILITY

All permanent underground structures shall be designed for durability in accordance with the minimum requirements stated in the following clauses.

8.5.1 Concrete Cover

8.5.1.1 For reinforced concrete structures, SS 544 shall be used to assess the exposure condition and durability requirement of the structures. The exposure class, ACEC-class, DC-class, cement composition and minimum requirement for concrete mix design shall be stated clearly in the durability assessment report and drawings.

Minimum exposure class of XC3 (corrosion induced by carbonation under moderate humidity or cyclic wet and dry condition) shall be allowed for in the design of all underground structures. When the structure is subjected to corrosion induced by chlorides, XD or XS classes shall also be considered.

8.5.1.2 Notwithstanding the cover required for durability, $c_{min,dur}$, derived from SS 544, the minimum allowance for deviation, Δc_{dev}, due to construction tolerance shall comply with the requirements in Table 8.4.

8.5.1.3 Also, the nominal cover c_{nom} to the outermost reinforcement shall be not less than 40mm for all permanent underground structural elements.

Table 8.4 – Minimum Requirements for Concrete Cover

Condition of Cast	$c_{min,dur}$	Δc_{dev}	$c_{nom} = c_{min,dur} + \Delta c_{dev}$
Directly against soil (e.g. diaphragm wall, SBP, pile foundation etc.) with exception of micropile	Refer to SS 544	50 mm 10 mm	 Min 40 mm
Against blinding or prepared ground (e.g. base slab)	Refer to SS 544	15 mm	Min 40 mm
Against formwork	Refer to SS 544	10 mm	Min 40 mm
Precast	Refer to SS 544	5 mm	Min 40 mm

8.5.2 Shrinkage and Thermal Cracking

8.5.2.1 Adequate consideration shall be given to the risk of early-age thermal cracking and shrinkage effects. Any requirements considered appropriate such as cement chemistry and curing methods shall be properly addressed as specified in the Materials and Workmanship Specification.

8.5.2.2 Crack widths due to early-age thermal cracking and shrinkage shall be calculated using SS EN 1992-3. In the case of early-age thermal cracking, reference shall be made to CIRIA C766 for structures designed to SS EN 1992. However, the beneficial effects of using cement replacement materials and other techniques such as internal cooling to control the heat of hydration may be taken into account provided that suitable evidence is presented to justify the magnitude of these effects.

8.6 FIRE RESISTANCE

8.6.1 With the exception of non-load bearing separation walls, all other elements of underground structures shall be designed and detailed for a 4-hour fire resistance, as specified in Singapore Civil Defence Force (SCDF)'s Code of Practice for Fire Precautions in Rapid Transit Systems.

8.6.2 To prevent spalling of concrete, methods specified in SS EN 1992-1-2 shall be adopted

8.6.3 In the case of non-load bearing separation walls, their fire resistance shall be determined by other requirements, such as fire compartmentation etc.

8.7 ANALYSIS

8.7.1 The structure shall be analysed for the actions specified herein to obtain the most severe combinations and envelopes of stress resultants on every component member.

8.7.2 The movements, both global and differential, of the structures shall be quantified from geotechnical analyses and considered in the design at all stages of construction and throughout the service life of the structure. In assessing the movements of the structures, due consideration shall be given to the movement of the ground, including settlement or heave as appropriate.

8.7.3 Complex structures shall be clearly identified in the Design Statement and analysed with grid analysis, finite element plate analysis, or similar, taking into account of three-dimensional behaviour. Such structures may include: -

a) Areas with irregular boundary conditions;
b) Areas where the action is not predominantly one-way;
c) Structures with unconventional geometry;
d) Non-vertical/Inclined elements; and
e) Areas where the out-of-plane action cannot be accurately modelled using plane frames, or any combination of these.

8.7.4 Other parts of structures with regular shapes (e.g. box) which are away from zones of three-dimensional effects may be analysed as plane frames.

8.8 DETAILING

8.8.1 Slabs and Walls

8.8.1.1 For the reinforcement layers nearest the concrete faces, the centre-to-centre spacing of reinforcement bars shall not be greater than 200mm and the clear spacing between bars shall not be less than 100mm, except at laps where it shall not be less than 75mm.

8.8.1.2 Particular attention shall be paid to the practicalities of the design and detailing of the slab to wall connections and the means by which the integrity of the construction joints at these connections will be assured. For external hull members, the connections between slabs and embedded walls such as diaphragm wall etc. shall be designed as rigid joints.

8.8.2 Columns

8.8.2.1 The spacing of the transverse reinforcement shall not exceed

- 300mm; or
- 20 times the minimum diameter of the longitudinal bars; or
- 0.75 times the effective depth of the section.

whichever is lesser.

8.8.2.2 The links shall be of grade 500 or 600 steel and the minimum bar diameter shall be 10mm, for column dimensions exceeding 500mm x 500mm or 600mm diameter.

8.8.2.3 The shear strength of concrete higher than strength class C50/60 shall be limited to that of C50/60.

8.8.3 Corner Details

Corner joints of large structural members, particularly with open moment, shall be detailed, in accordance with PD 6687-1 Annex B.3.

8.8.4 Construction Joints

The design and detailing shall be such that the number of construction joints will be as few as practicable. The suggested location of construction joints shall be indicated on the drawings.

8.8.5 Detailing of Shear Links

Where shear links are provided they shall enclose all tension reinforcement.

8.9 ADDITIONAL REQUIREMENTS FOR TRAINWAYS IN CUT-AND-COVER TUNNELS AND STATIONS

8.9.1 Size of Tunnels

With the exception of cells containing sidings per Clause 8.9.5, the size of each cell of a cut-and-cover tunnel shall accommodate the various items listed in Clause 7.5.2. Items (c), (g) to (l) can be accommodated in spaces 300mm wide on the walkway side of the tunnel and 350mm wide on the opposite side of the tunnel as shown on the Authority's Drawings.

8.9.2 Cross Passageways

Cross passageways between two independent single-bound cut-and-cover tunnels shall conform to the requirements of Chapter 7.

8.9.3 Drainage Sumps

The requirements of Chapter 11 shall apply for sumps within the lengths of cut-and-cover tunnel.

8.9.4 Ventilation

Tunnel ventilation regime shall be maintained by the use of jet fans mounted in the roof of the cut-and-cover tunnel. Vertical enlargements in the structure shall be provided for these fans. The size and extent of the structural enlargement and clearance requirements to any overhead conductor envelope shall be determined in co-ordination with the relevant System-wide Contractors.

8.9.5 Cells Containing Sidings

The minimum internal width of cells for sidings shall accommodate:

a) The Static Load Gauge, increased as necessary for Vehicle Throw;

b) 800mm minimum clearance between the Static Load Gauge (increased for Vehicle Throw) and service zones, to accommodate access at track level to a stationary train; and

c) 350mm wide service zones on each side.

8.9.6 First Stage Concrete

The first stage concrete is defined as the concrete needed to fill the gap, if any, between the top of the structural slab/tunnel lining and the underside of the track concrete placed by the trackwork contractor. It would therefore not be necessary to provide a separate layer of first stage concrete if the top of structural slab matches the level of the underside of the second stage (track slab) concrete. Where first stage concrete is required, the minimum thickness of the first stage concrete shall be 100mm. The minimum concrete strength class for the first stage concrete shall be C30/37.

8.9.7 Concrete Finish at Interfaces between Trainway Structure, First Stage and Second Stage Concrete

The concrete finish in all trainways at the interface between the trainway structure and the first stage concrete or the second stage concrete (if no first stage is used) and between the first and second stage concrete (where both are used) shall be an exposed aggregate finish. This requirement shall be shown on the design drawings.

8.10 ADDITIONAL REQUIREMENTS FOR ROAD TUNNELS, VEHICULAR UNDERPASSES AND DEPRESSED CARRIAGEWAYS

8.10.1 A minimum headroom clearance of 5.4 metres above all roadways shall be maintained across the full width of each carriageway. This applies to both permanent and temporary structures.

8.10.2 The road tunnels, vehicular underpasses and depressed carriageways shall be designed to allow for positioning and housing of electrical lighting fixtures, mechanical ventilation system, if any, and openings for cabling works.

8.11 PROVISION FOR FUTURE DEVELOPMENT

8.11.1 General

The items in the following Clauses shall be addressed in the design and shall be shown clearly on drawings. There shall be supporting narrative in the Development Interface Report.

8.11.2 Knock-out Panels for Access to Future Developments

Where access is required to future developments, appropriate provisions for the future openings (generally described as "knock-out panels") shall be made in the structure. In particular, the analysis, design and detailing of the structure shall allow for the opening being provided in the future. In addition, appropriate trimming reinforcement shall be provided. When such knock-out panels are provided, they shall be designed to facilitate future removal without compromising the structural integrity or watertightness of the structure.

8.11.3 Fire Separation for Railway Structures

A 4-hour fire separation shall be provided between railway and development areas unless specified otherwise in the SCDF's Code of Practice for Fire Precautions in Rapid Transit Systems. The minimum thickness of reinforced concrete walls or slabs separating railway and development areas shall be 200mm provided that the area of steel relative to that of concrete exceeds 1%.

8.11.4 Actions from Future Known Development

8.11.4.1 Unless indicated otherwise in the Particular Specification, cut off level of all piles and column stumps shall be 2.5m below finished ground level. Pile/column stump reinforcement shall protrude from the cut off level for a lap length and be protected by strength class C16/20 capping concrete. Alternatively, where soil cover is not available, suitably protected reinforcement couplers may be used, in which case the cut off level shall not be less than 1m below finished ground level.

8.11.4.2 The structural capacity and predicted settlement/deflection of piles and column stumps at the cut off level corresponding to the appropriate limit state shall be calculated and shown on pile/column stump capacity plans. The design shall demonstrate that the settlement limits specified in Chapter 6 will not be exceeded. Structural capacity shall be specified in terms of maximum horizontal force, vertical force and moment that can be applied at the cut off level of the pile or column stump.

8.11.5 Design Assumptions and Construction Constraints

8.11.5.1 The design shall take into consideration all requirements in the Development Interface Report as specified in General Specification and Particular Specification.

8.11.6 Protection for Waterproofing Works Interfacing with Future Development

Sufficient protection to the waterproofing works of underground structures shall be provided to allow for future construction above and adjacent to the underground structures. Detailed waterproofing drawings at the interface with future development and the protection requirement shall be included in the Development Interface Report and as-built drawings

8.12 WATERPROOFING

The ground water seepage rate for completed underground structure shall not exceed a general value of 0.12 litres per day per square metre ($l/d/m^2$). For any 10 square metres of hull structure the seepage rate shall not exceed 0.24 litres per day per square metre ($l/d/m^2$).

CHAPTER 9

ABOVE-GROUND STRUCTURES

9.1 SCOPE

This chapter covers the design requirements for above-ground structures that include:

a) above-ground stations and trainways;

b) vehicular bridges;

c) cycle and pedestrian bridges;

d) bus shelters;

e) covered linkways; and

f) road portal and gantry structures.

9.2 STANDARDS AND CODES OF PRACTICE

The design shall comply with the latest Singapore's adoption of Eurocode (SS EN), their National Annexes, Interim Advice Notes by Highways England, and the additional requirements herein.

9.3 ACTIONS

9.3.1 All design actions shall be in accordance with Chapter 3.

9.3.2 The structure shall be analysed for the specified actions to obtain the most severe combination of actions on every component member. The method and sequence of construction shall be clearly specified and taken into account in the design. Actions and combination of actions considered in the design shall include those occurring during construction stage.

9.3.3 The structurally acceptable margins of tolerance as specified in the Material and Workmanship Specifications shall be clearly specified for critical members and operations.

9.4 DESIGN REQUIREMENTS

9.4.1 General

The following design requirements shall be adopted:

a) A minimum headroom clearance of 5.4m shall be maintained across all roads for all structures.

Where the viaduct structure is running parallel to and above at-grade carriageways, the minimum headroom clearance of 10m shall be maintained.

The headroom clearance across Drainage Reserves, utility corridors, etc. shall comply with the requirements of the relevant authorities and utility agencies.

b) A minimum clear horizontal separation of 2.0m between structures of adjacent bridge decks shall be provided unless otherwise agreed by the Engineer.

c) Columns/Piers

(i) The assumed construction method and sequence shall be considered in the design of column and pier for temporary and permanent stages. The sway in any direction at the top of the column-head and/or crosshead during erection shall not, in any way affect construction (e.g. successful launching of pre-cast beams). This shall be clearly indicated in the design drawings. The design shall be revised should there be any deviation to the assumed construction method and sequence.

(ii) The longitudinal sway at the top of column-head and/or crosshead under the applied transient longitudinal forces (e.g. braking forces) shall not exceed an extent that will in any way affect the performance of bearings, movement joints and trackworks, etc.

d) No half-joints are permitted. The superstructure shall be fully supported with minimum two bearings on crossheads and abutments at movement joint locations.

e) Spacing of movement joints between a continuous frame shall be about 160m. All proposed movement joints shall be installed across the full width of the structures. Any deviation from this requirement due to site constraint shall be subjected to the acceptance of the Engineer. For bridge/viaduct supporting railway tracks, the movement joints shall not be located at the turnout location. In addition, the designer shall carry out track-structure interaction analysis as stated in Chapter 4 to verify the compatibility of bridge/viaduct and trackwork designs.

f) Inspection access

Access opening from the soffits to the voids in box girders shall be provided in each span to facilitate future inspection and maintenance. These access opening: -

(i) shall not be less than 800mm diameter,

(ii) shall preferably be located away from carriageways,

(iii) shall be covered with an easily removable access cover to the void. The cover shall be detailed such that it will stay in place.

Through access shall also be provided in each cell of a continuous box girder at diaphragm locations.

g) Security measures for Rapid Transit System (RTS)

Infrastructures that are to be built within 20m measured laterally from the edge of RTS viaduct or trainway shall be installed with a 3.5m (minimum) high security fence or other appropriate security measures to prevent items from being lobbed onto the viaduct or trainway.

h) Long Term Delayed Strain Estimation

Safety factor for long term extrapolation of delayed strains of 1.25 shall be applied to Ultimate Limit State design and the calculation of movement due to creep and shrinkage for bearings and movement joints design.

9.4.2 Prestressed Concrete

9.4.2.1 General

9.4.2.1.1 The maximum and minimum grades of concrete shall be strength class C50/60 and C32/40 respectively.

9.4.2.1.2 All assumptions made in the determination of the design prestress forces, e.g. vertical and horizontal curvature, friction and wobble, anchorage slip/draw-in loss shrinkage and creep of concrete, elastic shortening, properties of concrete and prestressing steel, etc. shall be clearly stated in the calculations and on the Drawings.

9.4.2.1.3 Prestressing anchorages shall be detailed such that they are easily accessible for inspection and maintenance. The detailing shall prevent the accumulation of water and dirt around the anchorage.

9.4.2.1.4 Additional blisters for future external tendons shall be provided for all box girder structures. The tendon forces shall be designed to compensate 30% of prestress loss at mid span and support of a span. The design, detailing and arrangement of additional intermediate blisters shall not be more than 12m apart.

9.4.2.1.5 Post-tensioning tendons shall have a minimum corrosion protection of Protection Level 2 in accordance with International Federation for Structural Concrete (fib)'s Bulletin 33.

9.4.2.2 Serviceability Limit State

a) Prestressed concrete elements with bonded or unbonded tendons for bridge and station structures shall be designed in accordance with Notes (D) and (F) of Table NA.2 NA to SS EN 1992-2.

b) At transfer and during construction stage, the flexural tensile stress in the concrete shall not exceed 1.0N/mm² due solely to prestress and co-existent permanent and temporary actions during erection.

c) Modification to Clause 5.10.2.1(1) and 5.10.3(2) SS EN 1992-1-1:

Immediately after anchoring, the force in the prestressing tendon shall not exceed 70% of the characteristic strength for both pre-and post-tensioning. The jacking force shall not exceed 75% of the characteristic strength during stressing operations unless written acceptance of the Engineer has been obtained.

d) Modification to Clause 7.2 (102) SS EN 1992-2

The compressive stress limit shall also be applicable to environment of exposure class XC.

9.4.3 Design Surface Crack Width

For reinforced concrete above-ground structures, the limiting calculated crack width for external elements exposed to weather and internal elements completely sheltered from rain shall not exceed 0.25mm and 0.30mm respectively for serviceability limit states characteristic combination. The following cases need not be considered: -

(i) Wind forces and thermal effects

(ii) SV196 under Load Model 3

In addition, Load Model 2 need not be considered except for cantilever slabs and the top flanges in beam-and-slab, voided slab and box-beam construction.

9.4.4 Vibrations

9.4.4.1 The design of bridge structures shall satisfy appropriate vibration serviceability requirements as specified in the relevant clauses of SS EN 1990, SS EN 1991 and their National Annexes.

9.4.4.2 In the selection of the structural framework for railway stations, careful consideration shall be given to the isolation or reduction of vibration transmitted from bridge structures to the station structures. Complete isolation shall be adhered to if practical.

9.4.5 Bridge Aesthetics

9.4.5.1 General

9.4.5.1.1 Due attention shall be given at the concept design stage to ensure that the completed structure is aesthetically pleasing. The guidelines in BA 41/98 shall be used.

9.4.5.1.2 It shall be demonstrated at concept design stage, and to the acceptance of the Engineer, that the design meets the requirements outlined below. The submission shall include the deck drainage layout, showing the locations of drainage pipes and how they are concealed from public view.

9.4.5.1.3 At least two photo-montages (view from different angles) to illustrate the final appearance of the bridge in relation to its surroundings shall be submitted to the Engineer for acceptance, as part of the concept design submission.

9.4.5.2 Member Sizes and Shapes

In sizing the structural members, the following conditions shall be satisfied: -

a) Any change in sectional depth shall be gradual.

b) No change in the shape of sections, other than that due to varying depth.

c) Column sizing shall give an appearance of a slender structure proportional to the superstructure. “Wall” like columns shall be avoided.

9.4.5.3 Treatment of Furniture

Due attention shall be given to the placement or treatment of furniture such as, down pipes, street lighting, electrical cables and irrigation pipes. The deck drainage layouts shall be such that drainage pipes, electrical cables and irrigation pipes could be strategically located and hidden from public view but with easy access for maintenance.

9.4.5.4 Long-Term Appearance and Detailing for Effective Water Management

9.4.5.4.1 Appropriate detailing to all structural elements to eliminate the chances of staining, concrete chipping off, etc. shall be clearly shown in the design for the Engineer’s acceptance. Such detailing shall include sloped surfaces, flashings and/ or drip grooves to direct water away from surfaces (e.g. parapet wall, crosshead, underpass portals) to prevent staining and chamfers to prevent concrete chipping.

9.4.5.4.2 Drainage pipes should also be protruded from vulnerable faces to direct water away to prevent staining.

9.4.6 Precast Concrete Segments

9.4.6.1 The following requirements shall be applied to the design of pre-cast concrete segments:

a) Dry joints are not allowed.

b) With epoxy glued joints at service stage, a minimum of 1.0N/mm^2 compressive stress across the whole section of the precast segment under all combination of actions shall be achieved. In addition, there shall be a minimum of 1.5N/mm^2 compressive stress across in-situ stitches of the precast concrete segments under all combination of actions.

c) During construction, a minimum compressive stress of 0.2N/mm^2 across any joint between segments shall be achieved and the difference between flexural stresses across the section shall be not more than 0.5N/mm^2.

9.4.6.2 For pre-cast segmental bridge design, where requirements are not covered in the SS EN and their National Annexes, the guidelines in the “AASHTO Guide Specifications for Design and Construction of Segmental Concrete Bridges” shall be adopted. Confinement reinforcement shall be provided for curved tendons in webs according to the AASHTO guide.

9.4.6.3 Shear key shall be provided for precast segment interfacing with cast in situ element

9.4.7 Foundation

9.4.7.1 The effects of foundation settlements on the structures, including movement joints and bearings shall be allowed for in the design. The settlements shall be within the allowable limits specified in Chapter 6.

9.4.7.2 Where piles are subjected to bending moment, the design comprising a single row of piles is not acceptable.

Piles supporting the superstructure directly (pile column) designed as frame system with the superstructure may be considered, subject to the Engineer’s approval.

9.4.7.3 The top of the footings/pilecaps shall be at least 1.0m below the finished ground level.

9.4.8 Bridge Abutments and Retaining Walls

9.4.8.1 Bridge Abutments

9.4.8.1.1 Reinforced-earth retaining structures shall not be used as bridge abutments that directly support the superstructures.

9.4.8.1.2 Abutments shall be designed against sliding and overturning and the effects of filling material in front of the abutment shall be neglected.

9.4.8.1.3 The characteristic earth pressure used for the design of the abutment walls shall be the “at rest” earth pressures.

9.4.8.2 Retaining walls

Base slabs of retaining walls including reinforced-earth retaining structures supporting vehicle and vehicular parapets shall be designed for vehicle collision forces in accordance with Clause NA.2.30.1 of NA to SS EN 1991-2.

9.4.9 Approach (Transition) Slab

An approach (transition) slab behind each bridge abutment or approach structure shall be provided to give a smooth transition from a rigid structure to a flexible structure as specified in Chapter 6.

9.4.10 Integral Bridges

Bridges with integral abutments shall be designed in accordance with PD 6694-1.

9.4.11 Assessment and Strengthening of Existing Bridge Structures not designed to Eurocode

9.4.11.1 For widening or assessment of existing bridge structures that were not designed to Eurocodes, the following requirements shall apply: -

a) All actions shall comply with Appendix 9.1;

b) Combination of actions shall comply with BD 37/01; and

c) Assessment of existing structural elements to BS 5400 and Appendix 9.2.

9.4.11.2 In the event that the structure capacity of the existing bridge structure is inadequate and strengthening is required, the strengthening and all new structural elements shall be designed to SS EN and Singapore National Annexes. The strengthening proposal shall demonstrate due consideration to the impact on drainage capacity, efficiency for the continued use of the structure during strengthening works, durability, compatibility with the existing bearings and overall articulation of the bridge structure, future maintenance/replacement and cost-effectiveness, etc.

9.4.11.3 All strengthening system shall be carried out by a specialist with the relevant experience and good track record. Fibre Reinforced Polymer (FRP) shall not be used unless otherwise accepted by the Engineer. The designs for steel plate bonding, external prestressing and FRP shall be in accordance with the standards in Table 9.1.

Table 9.1 – Design Standards for Strengthening System

Types of Materials	Design Standards
Steel plate bonding	BA 30/94
External prestressing	SS EN 1992 and its National Annexes
Fibre Reinforced Polymer (FRP)	TR55 - Design Guidance for Strengthening Concrete Structures using Fibre Composite Materials

9.5 BEARINGS

9.5.1 General

9.5.1.1 Bridge bearings shall always be in compression under all combination of actions.

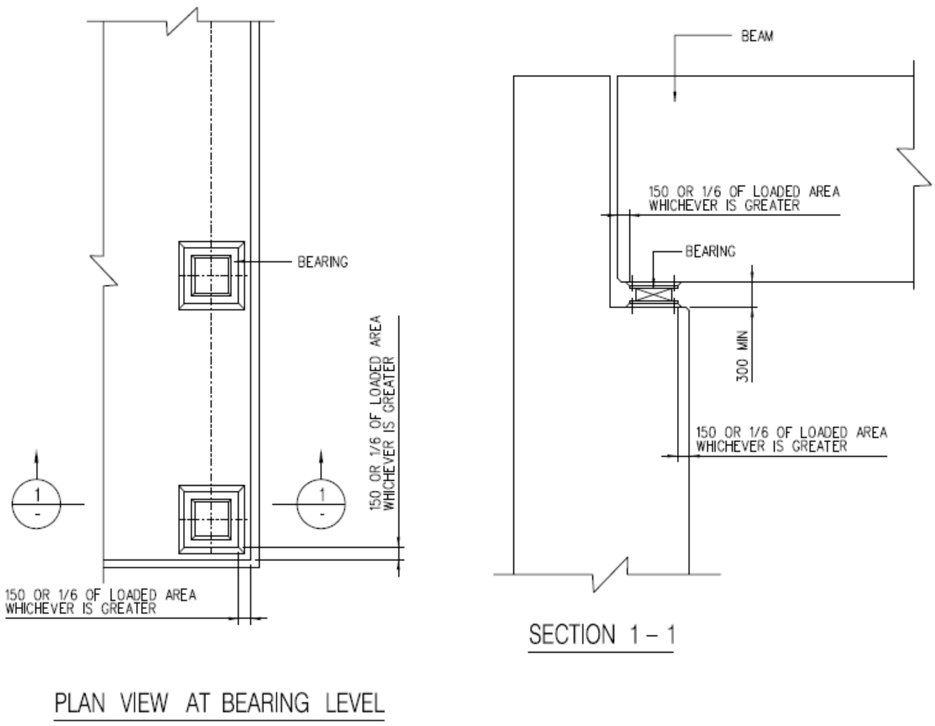
9.5.1.2 The manufacture, installation and performance of the bearings shall comply with Chapter 15 of the M&W Specification.

9.5.1.3 Structures shall be designed such that the number of bearings is minimised. The design shall take into account the need for easy inspection, maintenance and replacement of bearings. Space for jacking and permanent access platform shall be provided and detailed in the drawing.

9.5.1.4 Bearings for railway bridges shall be designed to accommodate the derailment loads specified in NA to SS EN 1991-2. The corresponding viaduct rotation under derailment loads shall be controlled to minimise damage to viaduct elements.

9.5.1.5 The position of bearings shall be such that the minimum clear distance from the edge of the bearing plate to the edge of concrete structure shall not be less than 150mm or 1/6 of the corresponding dimension of the loaded area measured in the same direction whichever is greater as shown in Figure 9.1.

Figure 9.1 – Viaduct Bearings



9.5.2 Bearing Inspection, Maintenance and Replacement

9.5.2.1 The design of structures shall consider future bearing inspection, maintenance and replacements with the following provisions: -

a) Provisions shall be made in the design to allow the bearings including their fixings to be inspected, repaired or replaced. The minimum height between the soffit of beam and top of crosshead/ abutment shall not be less than 300mm.

b) Minimum headroom clearance of 1800mm between soffit of beam and finish ground level shall be provided at abutment. Proper access to the abutment shall be provided.

c) Reinforced concrete maintenance platforms from both side of crossheads shall be provided with minimum provision of 800mm(W) and 1800mm(H) clear height between soffit of girder to top of concrete platform with stainless steel railing as illustrated in Figure 9.2 and Figure 9.3 for bearing inspection, maintenance and replacement where the bearings are inaccessible (e.g. above at-grade roads, water body or other structures)

d) Inspection gallery with structure clearance of 1000mm(W) and 1800mm(H) shall be provided below the deck at all movement joint locations as illustrated in Figure 9.3.

e) Designated jacking points for future bearing replacement shall be shown on drawings and permanent marking points shall be installed on the crossheads/abutments

Figure 9.2 – Cross-section of Pier at Movement Joint

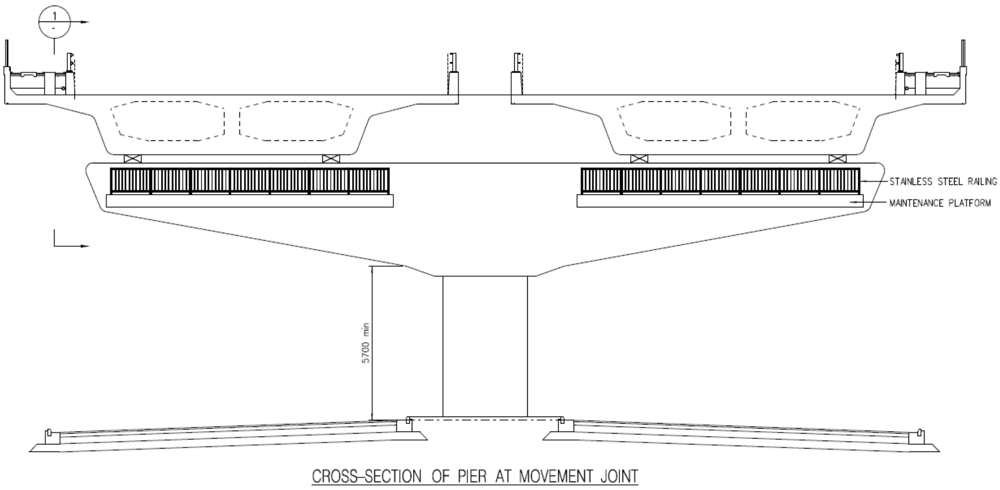
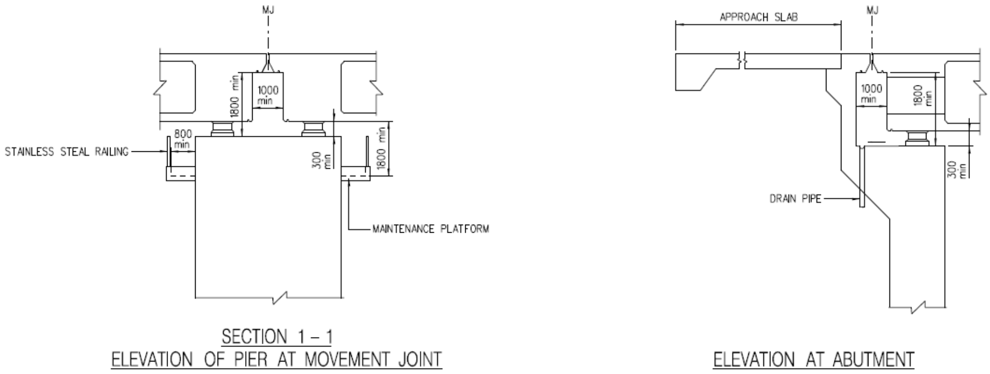


Figure 9.3 – Maintenance Platform with Inspection Galleries



9.5.2.2 Drawings showing the jacking point for the replacement of bearings shall be produced as part of the design drawings.

9.6 MOVEMENT JOINTS

9.6.1 General

9.6.1.1 For vehicular bridges, only elastomeric in metal runners joints (modular joints), cantilever comb or tooth joints and finger plate joints are acceptable. Such joints shall be heavy-duty surface mounted mechanical system with bolts and reinforcement embedded in the bridge deck. They shall be provided along the full width of the bridge deck including the parapets, maintenance accesses and footpaths, strictly in accordance with the manufacturer's recommendations. The installation of movement joints shall be carried out as close to the commissioning of the bridge as possible to reduce the effect of creep and shrinkage.

9.6.1.2 Structures shall be designed such that: -

a) The number of movement joints is minimised.

b) Movement joints with total longitudinal movement capacity exceeding 100mm shall not be used.

9.6.1.3 The movement joints shall be designed in accordance with BD 33/94 and BA 26/94.

9.6.1.4 Movement joints in railway stations shall be located so as to avoid damage to the architectural finishes. They shall accommodate shrinkage, creep and thermal effects. They shall be designed to be easily maintained and replaceable.

9.6.1.5 The Contractor shall submit the details of movement joint to the Engineer for acceptance. The details and location of movement joints, the size, length and spacing of the holding-down bolts shall be included. This shall be substantiated by calculations that include: -

a) Design movement ranges of the proposed joints.

b) Derivations of the resulting loads generated by the joints in accommodating these movements.

9.6.1.6 Detailed shop drawings, clearly indicating all dimensions, sections, longitudinal and transverse gradients of the joints in relation to the structure shall be submitted to the Engineer for acceptance.

9.6.2 Requirements

Movement joints shall satisfy the following requirements: -

a) They shall be either completely sealed to prevent ingress of water and granular material, or alternatively, designed to drain away water and granular material and prevent them from accumulating at the joint.

b) They shall be designed to enable easy maintenance and parts liable to wear shall be easily replaceable.

c) They shall be designed to accommodate shrinkage, creep, thermal effects, live loads and settlement.

d) They shall be designed so that the passage of pedestrians and cyclists is not impeded. The size of any open gap on the riding surface of the joint shall not exceed 65mm. Where pedestrians have access, a non-slip cover plate shall be provided.

e) They shall not unduly impair the riding quality of the surrounding surface for vehicular traffic, nor shall the passage of vehicular traffic cause undue noise or vibration.

f) For rail viaduct, no movement joint is allowed at turn-out areas. The design shall be coordinated with System Wide Contractor (Trackwork) and comply with Trackwork requirements.

9.7 ADDITIONAL REQUIREMENTS FOR PEDESTRIAN OVERHEAD BRIDGES AND HIGH LEVEL COVERED LINKWAYS (HLCL)

9.7.1 Stability Check

The pedestrian overhead bridge deck and HLCL overcrossing arterial road shall be checked for an impact force of 85kN applied in any direction between the horizontal and vertical on any part of the surface of the bridge or linkway. This check shall be considered for the construction and permanent stage, and even if the clearance height of the bridge is more than 5.7m.

9.7.2 Provision of Dowel Bars Joining Precast Bridge Deck and Pier

Where dowel bars are provided at support locations to connect the pre-cast bridge deck and pier, slot holes provided on the pre-cast beam shall be dimensioned taking into account short-term shrinkage, creep and elastic shortening of the pre-cast beam. Construction tolerances to facilitate launching operations shall also be taken into consideration. All dowel bars shall be hot dip galvanized.

9.7.3 High Level Covered Linkway

The HLCL shall be fabricated off-site, and installed on-site with bolted connections only.

9.8 WATERPROOFING AND IRRIGATION SYSTEM FOR FLOWER TROUGH AND PLANTING/ TURFING AREAS

A waterproofing agent shall be applied to the inner surfaces of flower troughs and planting/turfing areas on structures as specified in Chapter 13 of the M&W Specification.

The design, supply and installation of the mechanical irrigation system for flower troughs shall comply with the requirements of the National Parks Board and other relevant authorities as appropriate. The hydraulic calculations shall be submitted to the relevant authorities for approval.

9.9 PARAPET SYSTEM FOR VEHICULAR AND PEDESTRIAN OVERHEAD BRIDGES

9.9.1 General

9.9.1.1 The parapet systems for bridges shall be designed for collision loads specified in Chapter 3.

9.9.1.2 Only 2-rail aluminium alloy railings mounted on a concrete parapet for Normal Containment vehicle parapet and a full height concrete parapet for Very High Containment parapet shall be used.

9.9.1.3 Normal Containment vehicle parapets and Very High Containment parapets shall have a minimum total height of 1.5m above the adjacent finished carriageway level.

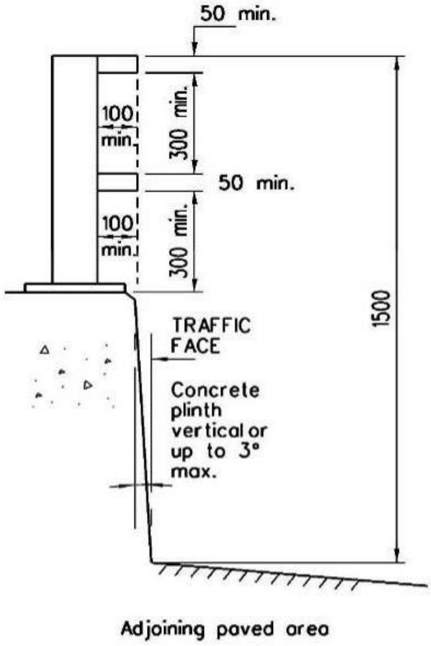
9.9.1.4 To calculate the vehicular impact force, the effective longitudinal member for the concrete panel alone shall be taken as 1.0 irrespective of the height of the panel for Normal Containment vehicle parapets.

9.9.1.5 Normal Containment Parapet

Except for bridges over high-risk locations such as RTS and railway lines, Normal Containment vehicle parapets shall be used for all vehicular bridges. The typical profile of the parapet shall be as shown in Figure 9.4 with the following requirements:

a) The traffic face of the concrete plinth vertical shall be up to maximum 3^{o}.
b) 75mm splayed raised kerb is not required except where integrated with continuous deck drainage system; and
c) Non-shrink grout with slopes at the edges shall be used for levelling the base plates of the Normal Containment railing posts. The grout shall not cover the base plates.
d) Railings on Normal Containment vehicle parapet including posts shall be of aluminium alloy. Pedestrian railings including posts shall be of aluminium or stainless steel.

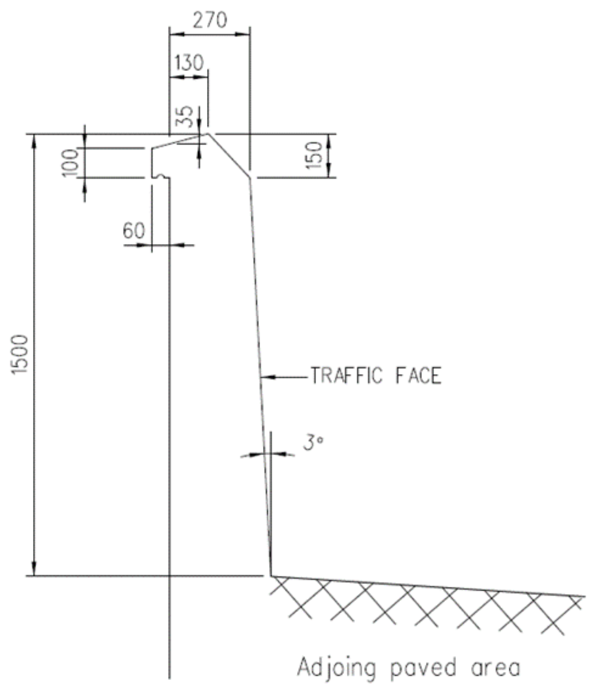
Figure 9.4 – Normal Containment Parapet



9.9.1.6 Very High Containment Parapet

Very High Containment parapets for vehicular bridges shall be used over high-risk locations such as RTS and railway lines. The typical profile shall be as shown in Figure 9.5.

Figure 9.5 – Very High Containment Parapet



9.9.1.7 Pedestrian Parapets

Pedestrian parapet shall be designed for forces specified in Chapter 3.

9.9.2 Additional Requirements for Vehicular Bridge Parapets

9.9.2.1 To prevent vehicles from striking the ends of vehicular bridge parapets, safety fences shall be provided in accordance with the Authority's Drawings.

9.9.2.2 The supports for gantries, directional and information signs, etc. shall be integrated with the bridge parapet. The shape of supports must blend in with the profile of the bridge and shall be subject to the approval of the Engineer.

9.9.2.3 Unless otherwise specified, no parapet wall shall be erected on structures along the centre median of non-expressways with dual carriageways. The centre median shall be formed with kerbs of Type K2 profile as shown in the Standard Details of Road Element.

9.10 THERMAL RAIL FORCES IN RAILWAY TRACKS

Provision shall be made for horizontal transverse and longitudinal forces due to temperature variation in the rail on railway bridges. The forces shall be applied in a horizontal plane at the top of the low rail as follows:

a) Transverse Force: The transverse force (T) per linear metre of structure per rail shall be determined by the following formula:

   T= 535/R (KN)

   Where R is the radius of rail curvature in metre.

b) Longitudinal Force: A longitudinal force of 180kN per rail shall be applied to the first 3 columns or piers adjoining any abutment or cross-over structure.

9.11 RAILWAY DECK FURNITURE, DRAINAGE AND WATERPROOFING

The railway deck furniture, drainage and waterproofing systems shall be designed for all effects and requirements of the railway including 25mm vertical lift for bearing replacement as specified in M&W Specification Chapter 15. The extra 10mm vertical lift is in addition to the 15mm limiting vertical lift for RTS bridge beams as specified in M&W Specification Chapter 15 and is the tolerance required for bearing replacement.

9.12 ELECTRICAL AND MECHANICAL REQUIREMENTS

Electrical and Mechanical (E&M) requirements must be considered in the development of all structural designs. Such consideration shall include the following:

a) The incorporation of stray current corrosion control systems by providing a continuous conductor for stray currents to return to the substations so as to reduce the impact of stray direct currents on structures.

b) Reinforcement in the plinth and deck designed to avoid interference with attenuation of the signalling circuits on structures.

c) The incorporation of an adequate water drainage system for all structures.

d) Special care shall be taken over the location of gullies at points and crossing areas of structures.

9.13 ROAD PORTAL AND GANTRY STRUCTURES

9.13.1 Road signs and signal gantry structures shall be designed in accordance with BD 51, collision loadings in Chapter 3 and the details shown in the Authority's Drawings, where applicable.

9.13.2 Collision protection beams and structures shall be designed in accordance with BD 65 and the details shown in the Authority's Drawings, where applicable. Collision forces shall be in accordance with Chapter 3.

APPENDIX 9.1

Assessment of Existing Bridge Structures not designed to Eurocode

A.9.1 HIGHWAY LOADS

A.9.1.1 General

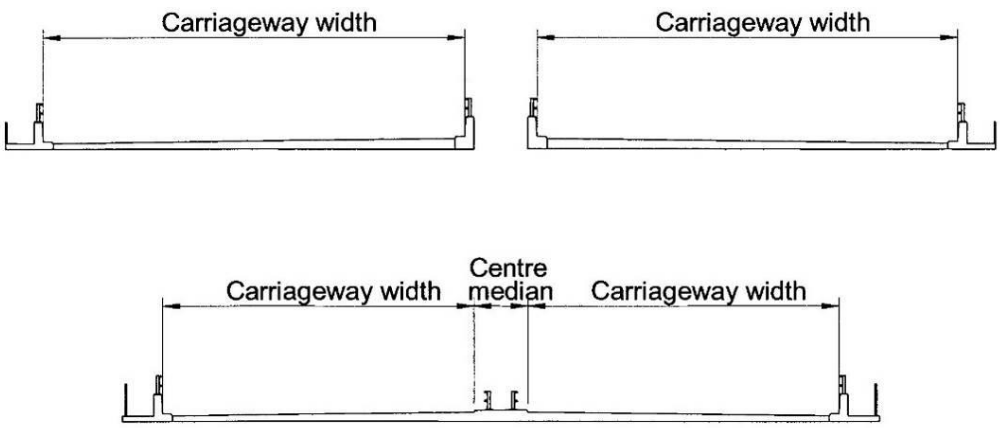
Highway loads shall comply with BD 37/01 except where otherwise specified below:

A.9.1.1.1 Carriageway

The carriageway shall be that part of the running surface which includes all traffic lanes, hard shoulders, hard strips and marker strips.

The carriageway width is the width between parapets or the width between parapets and raised kerbs of centre median, as shown in Figure A.9.1. The carriageway width shall be measured in a direction at right angle to the line of parapets, lane marks or edge marking.

Figure A.9.1 – Carriageway Width



A.9.1.2 Live Loads

A.9.1.2.1 Highway live loads shall comply with BD 37/01, except for the following:

a) The HA Uniformly Distributed Load (HA-UDL) shall be as given below:

(i) For loaded lengths from 2m up to and including 50m

$W = 403\ (1/L)^{0.67}$

(ii) For loaded lengths in excess of 50m but less than 1600m

$W = 43\ (1/L)^{0.1}$

Where L is the loaded length in metres and W is the load per metre of notional lane in kN. For loaded lengths above 1600m, the UDL shall be agreed with the Engineer.

(iii) For application of type HA-UDL and HA Knife Edge Load (HA-KEL), at least two lanes shall have a lane factor of 1.0 and the other lanes shall have lane factors of 0.6.

b) HA Wheel Load

In addition to the single wheel load of 100kN specified in BD 37/01, a separate load case of 2 numbers of 120kN wheel loads placed transversely, 2m apart, shall also be considered in the design for local effects.

c) HA Longitudinal Traction or Braking force

The nominal HA longitudinal traction and braking force shall be 10kN/m applied to an area one notional lane wide multiplied by the loaded length plus 250kN, subject to a maximum of 850kN.

d) Lateral loads on piers

Piers shall also be designed for a minimum horizontal force of 5% of the nominal vertical loads that consist of the permanent vertical load and ⅓ Type HA loading on one notional lane in each direction.

This horizontal force shall be multiplied by partial load factors of 1.0 for the serviceability limit state and 1.4 for ultimate limit state. It shall be applied longitudinally and transversely at the footing/foundation level as two separate loading conditions.

e) HB Loading

All structures shall be designed for HB loadings as follows;

Table A.9.1 – HB Loading

<table>
<tr><th>No. of units of HB Loading</th><th>Loading application</th></tr>
<tr><td>30 units
(HB - 30)</td><td>To be applied in co-existence with the relevant HA loading in accordance with BD 37/01</td></tr>
<tr><td rowspan="2">45 units
(HB - 45)</td><td>To be restricted to the centre 5m strip of the carriageway with no traffic on all other lanes except at the following areas where HB-45 is free to travel anywhere between the parapets:
(i) Along slip roads or loops of the interchange or flyover with no other associated loading on the structure.
(ii) For 80m length of the main structure prior to the approach to the slip road or loop with no other associated loading on the structure, as shown in Figure A.9.3.</td></tr>
<tr><td>Where two separate carriageways are supported on one structure, the loading shall be not less than:
(i) One number of HB-45 loading at any one time; and
(ii) Type HA loading to be also applied on the other carriageway as illustrated in Figure A.9.4, where inclusion is more critical. At least two lanes shall hav a lane factor of 1.0 and the other lanes, lane factors of 0.6.</td></tr>
</table>

Figure A.9.2 - Zone in which HB-45 is free to travel before the approach to the slip road or loop

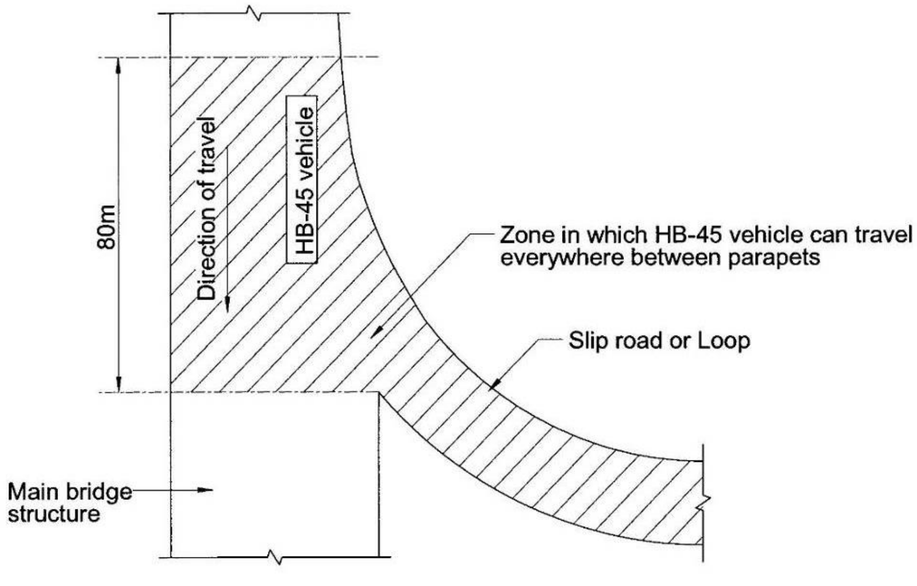
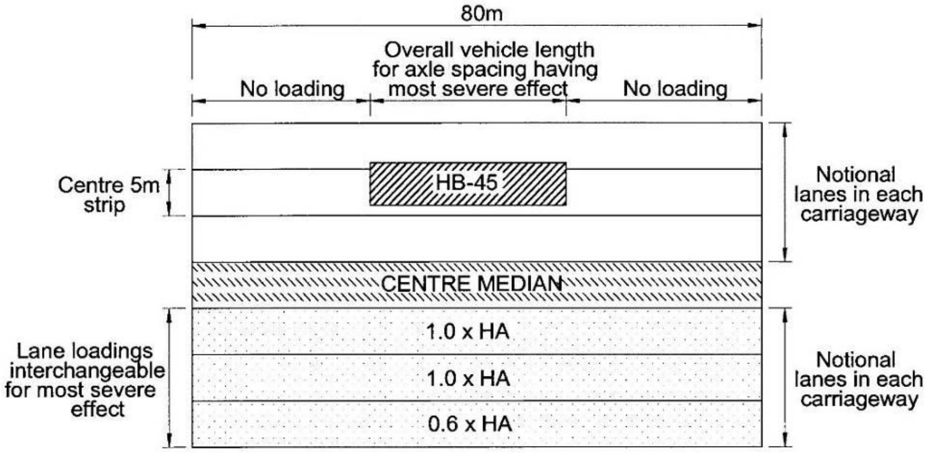


Figure A.9.3 - An example of HB-45 with HA loading for single structure carrying two separate carriageways



A.9.1.2.2 Highway Loads on Members Spanning Transversely

Where structural members (e.g. beams, slabs, etc.) span transversely and where it is not possible to determine the effective loaded lengths for HA-UDL, the loading given in clause A.9.1.2.1 shall be applied, subject to the following modifications:

Table A.9.2 – Highway loads on members spanning transversely

Loadcase	**Loading application**
HA Loading	20kN/m^2 shall be applied in place of A.9.1.2.1 (a). At least two lanes shall have a lane factor of 1.0 and the other lanes shall have lane factors of 0.6. HA-UDL in co-existence with HB-30 loading need not be considered
HB – 45 Loading	Where two separate carriageways are supported on one structure, the loading shall be not less than: (i) One number of HB-45 at any one time; and (ii) 20kN/m^2 to be also applied to the other carriageway, where inclusion is more critical. At least two lanes shall have a lane factor of 1.0 and the other lanes, lane factors of 0.6.
Traction and braking force	10kN/m shall be applied to an area one notional lane wide multiplied by the length of the structure between movement joints, subject to a maximum of 850kN.

For the above load cases, γ_{fL} to be considered in load combinations is given in Table A.9.3. γ_{f3} shall be applied in accordance with relevant code requirements.

Table A.9.3 – Load factors γ_{fL} to be used in load combinations

Limit State	**γ_{fL} to be considered in combination**				
	1	2	3	4	5
ULS	1.30	1.10	1.10	-	-
SLS	1.10	1.00	1.00	-	-

A.9.1.2.3 Highway Loads for Underground Structures

A.9.1.2.3.1 Structures under roads shall be designed for the following loadings:

Table A.9.4 – Highway Loads for Underground Structures

Depth of cover above top of structure roof slab level	**Highway loads**
≤ 600mm	Full vehicular live loads specified in Clause A.9.1.2.1 and A.9.1.2.2 as appropriate.
> 600mm	The more critical of: (i) HA wheel load and HB loading (specified in Clause A.9.1.2.1 and A.9.1.2.2 as appropriate). (ii) HB loading (as specified in Clause A.9.1.2.1).

The structures shall be designed for the following superimposed dead loads:

a) The top 200mm of cover shall be considered as surfacing. γfL for superimposed dead load (premix) shall be applied in accordance with BD 37/01.

b) An additional uniformly distributed load of 5kN/m2 shall be applied. γfL for superimposed dead load (others) shall be applied in accordance with BD 37/01.

A.9.1.2.3.2 In the case of underground structures serving road vehicles (e.g. vehicular underpass and road tunnels), highway loads inside and on top of the vehicular underpass shall be assumed to co-exist, with the exception that only HB-45 loading needs to be considered for any given load combination.

The HB-45 shall be placed anywhere over and through the structure with co-existing HA loading on that carriageway, where more onerous. HA loading shall be simultaneously applied on all the other carriageways, where more onerous.

Where it is not possible to determine the effective loaded lengths for HA loading, the highway loads, over and through the underground structure shall be in accordance with clause A.9.1.2.2.

A.9.1.2.3.3 Dispersal of Loads

a) The HA-KEL may be dispersed through the surfacing and fill from the depth of 200mm below the finished road level at 1 horizontally to 2 vertically to the top of the structural slab of underground structures.

b) Wheel loads may be dispersed through the surfacing and fill from the finished road level at 1 horizontally to 2 vertically to the top of the structural slab of the underground structure

A.9.1.3 Wind Loads

Approximate loads for the preliminary design of internal walls, claddings and false ceilings in road tunnels are given in the table below. These loads shall be verified by the relevant Specialist Contractor and the design adjusted accordingly.

Elements	Wind loads and Requirements
Elevated road related structures	The mean hourly wind speed shall be 20 m/s. Other recommendations of BD 37/01 on the computation of wind forces shall be adopted.
Internal walls, claddings and false ceilings in road tunnels 600mm	Nominal loads shall be ±1.5 kN/m^2, applied in the combinations for wind loads as defined in BD 37/01. For fatigue on metal elements of cladding, the stress cycle values shall be taken as 50 million.

A.9.1.4 Collision Loads

The collision loads shall be applied in accordance with Table A.9.5.

Table A.9.5 – Collision Loads

<table>
<tr><th colspan="2">Description</th><th>Expressways</th><th>Other Roads</th></tr>
<tr><td colspan="2">Road and railway bridge sub-structures located within Road Reserve or less than 4.5m from edge of carriageway and superstructures ≤ 5.7m clear height</td><td rowspan="3">BD 60/94</td><td>BD 60/94</td></tr>
<tr><td colspan="2">Pedestrian Overhead Bridge sub-structures located within Road Reserve or less than 4.5m from edge of carriageway</td><td>BD 37/01</td></tr>
<tr><td rowspan="2">Pedestrian overhead bridge superstructures</td><td>Clear height ≤ 5.7m</td><td>* 50kN</td></tr>
<tr><td>Clear height > 5.7m</td><td>* 50kN</td><td>* 50kN</td></tr>
</table>

* Nominal load to be applied in accordance to Clause 6.8.2 of BD 37/01

Collision loads shall be considered even if vehicular impact guardrails are provided or even when there is no vehicular access to column positions.

Bridge piers situated in navigational channels shall be designed for ship collision loads. The loads shall be subject to approval by the relevant authorities. Protection system against such collision loads shall be considered.

A.9.1.5 Loads on Parapets and Railing

Parapets and railing for road vehicle containment purposes shall be designed in accordance with the requirements in Chapter 3.

A.9.2 PEDESTRIAN LOADS

For structures serving pedestrians, the following loads shall be used:

Table A.9.6 – Loads for structures serving pedestrians

<table>
<tr><th colspan="2">Types of Loadings</th><th>Nominal loads (kN/m^2)</th></tr>
<tr><td colspan="2">Pedestrian live load</td><td>5</td></tr>
<tr><td rowspan="2">Roof structures or provision for future installation of roof structures</td><td>Dead Load</td><td>0.5</td></tr>
<tr><td>Live Load</td><td>0.5</td></tr>
</table>

A.9.3 HIGHWAY LOADS ON TEMPORARY DECK STRUCTURES

The highway loads on temporary deck structures shall be not less than the following:

a) HA loading as given in BD 37/01. The HA uniformly distributed load specified in Clause A.9.1.2.1 (a) is not applicable.

b) 25 units of Type HB loading.

c) Loading from construction vehicles. Any limits on construction vehicle loading shall be clearly indicated on the Temporary Works drawings.

The load factors given in Table 1 of BD 37/01 and γf3 shall be applied in accordance with code requirements. There shall be no reduction in load factors or material factors.

A.9.4 TEMPERATURE LOADS

Temperature loads shall be considered in accordance with the following:

a) Shade Air Temperature Range:

The shade air temperature range shall be taken as ±10°C from a mean temperature of 27°C. For these shade air temperatures, the extreme maximum and minimum mean temperature in concrete structures given in Table A.9.7 shall be adopted.

b) Temperature in Combination with Wind Force:

Where forces due to changes in temperature are considered in combination with maximum wind forces, the temperature range for all types of structure shall be taken as 27°C ± 5°C.

c) Temperature Gradient:

The effects of local strains from temperature gradients within the concrete structure and parts of the structure shall be calculated from the values of maximum temperature differences given in Table A.9.7 and Figure A.9.5. The effects of temperature gradient need not be considered in combination with maximum wind force.

d) Coefficient of Thermal Expansion:

The coefficient of thermal expansion for 1.0°C shall be taken as 12x10–6 for steel and concrete.

e) Temperature Range for movement joints and bearings:

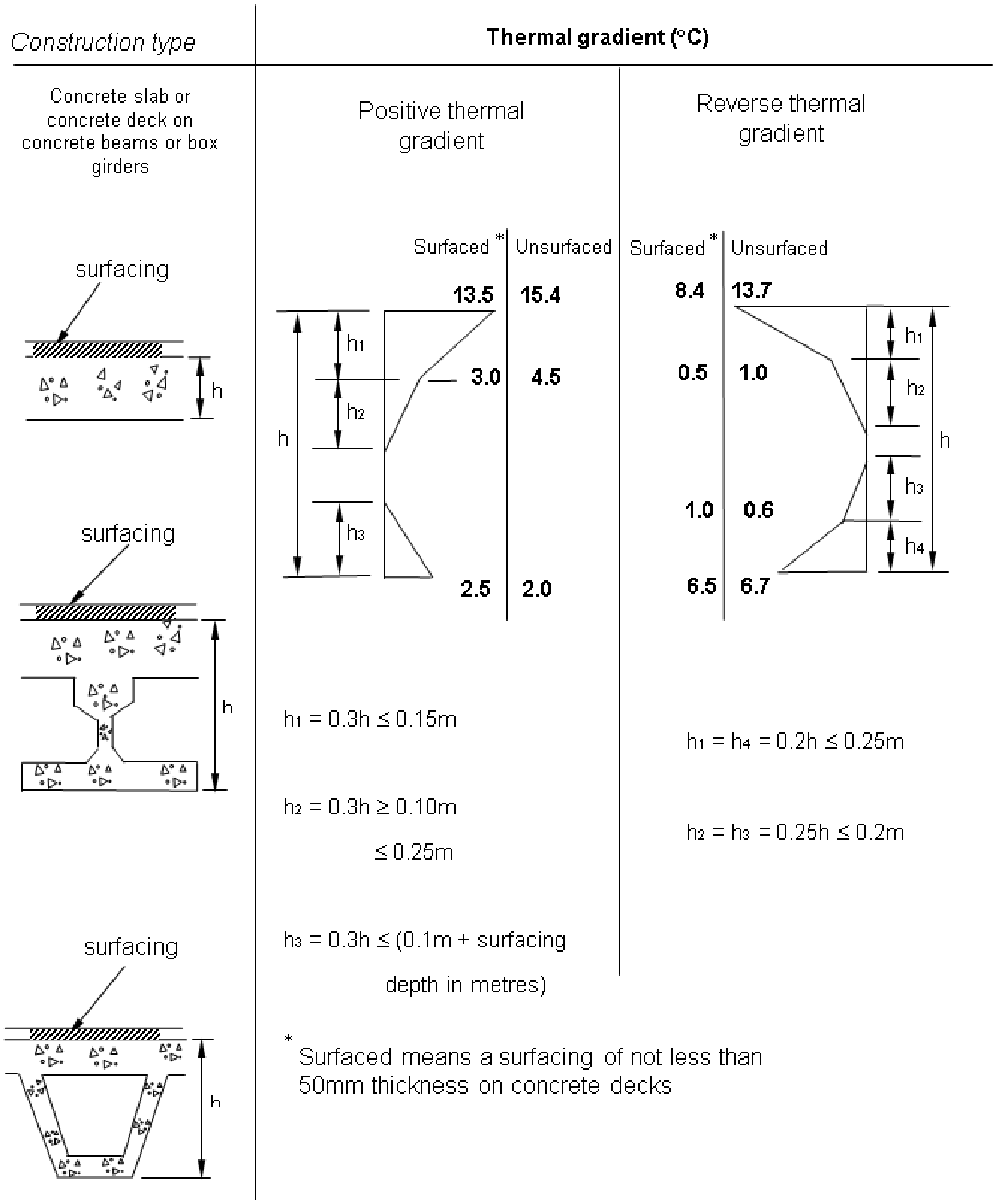
Bridge bearings and movement joints shall be designed for the temperature range given in Table A.9.7.

Table A.9.7 – Temperatures in concrete beam or box girders

Maximum and Minimum Mean Bridge Temperature (°C)		**Maximum Temperature Difference (°C) (See Figure A.9.5)**		**Maximum Reversed Temperature Difference (°C) (See Figure A.9.5)**	
Surfaced and Un-surfaced		Surfaced	Un-surfaced	Surfaced	Un-surfaced
Maximum	Minimum				
35	21	13.5	15.4	8.4	13.7

Note: Surfaced means a surfacing of not less than 50mm thickness on concrete decks

Figure A.9.5 – Temperature Gradients



APPENDIX 9.2

Assessment of Existing Bridge Structures not designed to Eurocode

Serviceability Limit State

Modification to clauses 4.2.2(a) and (b) of BS 5400: Pt.4 -

a) Prestressed concrete elements of bridge structures shall be designed as Class 2 under all load combinations, except under dead load and superimposed dead load, where they shall be designed as Class 1.

b) Modification to clause 6.3.2 of BS 5400: Pt.4 -

For members that are both pre- and post-tensioned (i.e. hybrid system), the tensile stresses shall not exceed the design flexural tensile strength of the concrete, which shall be taken as $0.36\sqrt{f_{cu}}$ and $0.36\sqrt{f_{ci}}$ at service and transfer respectively.

c) Modification to Clause 6.7.1 of BS 5400: Pt.4 -Maximum Initial Prestress:

Immediately after anchoring, the force in the prestressing tendon shall not exceed 70% of the characteristic strength for both pre-and post-tensioning. The jacking force shall not exceed 75% of the characteristic strength during stressing operations unless written acceptance of the Engineer has been obtained

CHAPTER 10

ROAD

10.1 SCOPE

10.1.1 This chapter covers the geometric design of all roads and the design requirement of all road related facilities.

10.1.2 The design shall comply with the specified desirable value. The need to use the absolute value must be demonstrated and is subject to the Engineer's acceptance.

10.1.3 Where the relevant design requirement cannot be achieved, due to site constraint or other reason, the potential safety hazard shall be identified, evaluated and the appropriate measures proposed to mitigate the risk associated with the hazard to achieve an acceptable risk level.

10.1.4 The details of road related facilities shown in the Authority's Standard Details of Road Elements shall be followed.

10.1.5 No part of any road related facility e.g. bus shelter, earth retaining structure, pier foundation etc. which are to be maintained by the Authority shall be constructed outside the road reserve.

10.2 DEFINITION

10.2.1 ***Bridge*** is a vehicular structure that over crosses an existing river, lake, sea, valley and utility reserve.

10.2.2 ***Crossfall*** is the slope, measured at the right angle to the alignment of the carriageway.

10.2.3 ***Design Speed*** is the nominal speed fixed to determine the geometric properties of a road.

10.2.4 ***Dual (x-lane) Road*** is a two directional road (x-lane in each direction) with a physical separator as a central divider (or) median.

10.2.5 ***Flyover*** is a vehicular structure that over crosses an existing road, rail or traffic junction.

10.2.6 ***Loop*** is a circular connecting road to allow vehicles to enter or exit an interchange from one level to another level.

10.2.7 ***Ramp*** is connecting road between two arms of an interchange.

10.2.8 ***Slip Road*** is the connecting road to allow vehicles to enter or exit the interconnecting carriageway and to bypass the intersection at-grade.

10.2.9 ***Tunnel*** is a below ground and/or below structures vehicular structure that is more than 90 metres in length.

10.2.10 ***Underpass*** is a below ground and/or below structures vehicular structure that is less than 90 metres in length.

10.2.11 ***Undivided (x-lane) Road*** is a bi-directional road with no physical separator/ divider with x-lane in total.

10.2.12 ***Viaduct*** is an elevated vehicular structure that is more than one kilometre in length.

10.3 CLASSIFICATION OF ROAD

The road category in relation to the class of road shall be as in Table 10.1.

Table 10.1

Class of Roads	Road Category
Expressway	1
Semi-Expressway	1A
Major Arterial	2
Minor Arterial	3
Primary Access	4
Local Access	5

10.4 HORIZONTAL AND VERTICAL ALIGNMENT

10.4.1 General Control

10.4.1.1 Horizontal Alignment

There are several general controls that shall be considered when designing the horizontal alignment.

a) Where it becomes necessary to introduce curve of lower standard than the design requirement, the respective design speed between the successive geometric elements shall not differ by more than 10 km/h;

b) Where the radius of a curve is below the design requirement, additional lane width shall be provided to accommodate heavy vehicle (refer to Figure 10.1) off-tracking within the curve;

c) Curve shall be sufficiently long to avoid the appearance of a kink. Curve shall be at least 150m long for a central angle of 5°, and the minimum length shall be increased by 30m for each degree decrease in the central angle;

d) Opening for vehicle turning at centre median along horizontal curve shall not be provided if the intersection sight distance is not achievable; and

e) on a horizontal curve, there shall be no obstruction i.e. longitudinal barrier, tree, shrub, embankment, other structure or building that restrict the driver's sight distance. If the required sight distance cannot be achieved, then lane widening shall be provided.

10.4.1.2 Vertical Alignment

There are several general controls that shall be considered when designing the vertical alignment: -

a) a smooth grade line with gradual changes shall be used instead of a line with numerous breaks and short lengths of grades;

b) grade through intersection shall not exceed 3% to avoid adverse effect to turning movement and ensure effective surface drainage;

c) a sag vertical curve or a flat section is desirable in advance of any channelization at an intersection and ramp take-off at an interchange, in order to provide sufficient sight distance; and

d) in providing sag vertical curves, one long vertical curve is more desirable and broken-back grade line shall be avoided.

10.4.1.3 Combination of Horizontal and Vertical Alignment

To avoid undesirable effect of poor combination of vertical and horizontal curve, the following principles shall be observed: -

a) the tangent point for both vertical and horizontal curve shall coincide;

b) when condition (a) cannot be met, the vertical curve shall be completely within the horizontal curve and have common mid-point. If the mid-point is unable to coincide, the distance between the mid-point of both curves shall be less than 0.25 times the length of the horizontal curve;

c) sharp horizontal curvature shall not be introduced near the top of a crest vertical curve, or near the bottom of a sag vertical curve;

d) the zero crossfall point within super-elevation development length shall not coincide with the bottom of a sag vertical curve; and

e) both horizontal and vertical curves shall be kept as long as possible and where both are present, they shall not be considered separately.

10.4.2 Horizontal Alignment

10.4.2.1 Design Speed

10.4.2.1.1 Main Carriageway

The geometric design requirement of road shall be as follows: -

Table 10.2

Parameter	**Road Category**					
	1	**1A**	**2**	**3**	**4 / 5**	**Special Needs Access**
Design Speed (km/h)	90	80	70	60	50	40
Desirable Minimum Radius (m)	355	270	195	145	95	65
Absolute Minimum Radius (m)	340	255	185	135	90	60
Desirable Maximum Super-elevation (%)	5	5	5	5	5	5
Absolute Maximum Super-elevation (%)	6	6	6	6	6	6

10.4.2.1.2 Interchange Ramp / Loop and Slip Road

The design speed for the interchange ramp/loop and slip road shall be as follows: -

Table 10.3

Design Speed (km/h)	**From Expressway to Expressway & From Semi-Expressway to Expressway**	**From Expressway to Semi-Expressway / Others**
Desirable	70	60
Absolute	60	50

10.4.2.2 Sight Distance

10.4.2.2.1 Stopping Sight Distance

The minimum stopping sight distance for various road gradient shall be determined by the following formula:

$$S = \frac{R_T\,V}{3.6} + \frac{V^2}{254\,(F + 0.01G)}$$

Where,

F = Longitudinal Friction Factor (see Table 10.4)

G = Longitudinal Grade (%)
(+ for up-grades and - for down-grades)

R_T = Reaction Time (2.5s for desirable, 2s for absolute)

S = Stopping Sight Distance (m)

V = Design Speed (km/h)

Table 10.4

Design Speed, V (km/h)	**40**	**50**	**60**	**70**	**80**	**90**
Longitudinal Friction Factor, F	0.56	0.52	0.48	0.45	0.43	0.41

10.4.2.2.2 Intermediate Sight Distance

a) Intermediate sight distance is equal to 2 times stopping sight distance.

b) Intermediate sight distance is only applicable to undivided 2-lane road.

c) Where intermediate sight distance is unachievable because of site constraint, the stopping sight distance shall be used and the appropriate control measures to prohibit overtaking shall be introduced.

10.4.2.2.3 Overtaking Sight Distance

The following sight distances are to be considered in the design. If overtaking sight distance for an undivided road cannot be achieved, the intermediate sight distance shall be adopted.

Table 10.5

Design Speed (km/h)	**Minimum Overtaking Sight Distance (m) for undivided 4-lane road**
40	150
50	200
60	300
Height of eye (m)	1.15
Height of object (m)	1.15

Note:

Overtaking sight distance is only applicable to undivided 2-lane road.

10.4.2.2.4 Intersection Sight Distance

The desirable intersection sight distance at unsignalised junction can be derived from the following formula and to be read in conjunction with Figure 10.2.

$$\mathrm{ISD} = \frac{\mathrm{R_T\ V}}{3.6} + \mathrm{S}$$

Where,

ISD = Intersection Stopping Sight Distance (m)

R_T = Reaction Time
(2.5s for desirable, 2s for absolute)

S = Stopping Sight Distance (m)

V = Design Speed (km/h)

10.4.2.2.5 Sight Distance on Horizontal Curve

On a horizontal curve where the obstruction on the inside of the curve cannot be removed, offset distance to the obstruction shall be provided. The offset distance can be derived from the following formula and shall be read in conjunction with Figure 10.3.

$$O = R\left[\left(1 - \cos\frac{28.65\ S}{R}\right)\right]$$

Where,

O = Offset (m)

R = Radius (m)

S = Stopping Sight Distance (m)

10.4.2.3 Horizontal Curve

The various types of horizontal curve used are as shown in Figure 10.4.

10.4.2.3.1 Simple Curve

This is an arc of constant radius which achieves the desirable deflection without using a transition curve. This type of curve shall be used whenever possible.

10.4.2.3.2 Compound Curve

A compound curve consists of two or more curves with deflections in the same direction immediately adjacent to each other. In general, the use of compound curve is not favoured. Where it is necessary, the following guidelines shall be applied: -

a) curve shall have radii greater than 1000m.

b) where radii less than 1000m have to be used, the design speed for each curve shall desirably be within 10km/h of each other.

c) on main road, the ratio of the radius of the flatter curve to the sharper curve shall not exceed 1.5:1; and

d) at junction intersection and ramp, the ratio of the radius of the flatter curve to the sharper curve can be up to 2:1.

10.4.2.3.3 Reverse Curve

A reverse curve consists of two curves of deflections in the opposite direction which are joined by a relatively short tangent distance. Reverse curve shall not be used unless there is sufficient distance between the curves to introduce full super-elevation of the two curves. In general, the use of reverse curve is not favoured. Where it is necessary, the following guidelines shall be applied.

a) the tangent distance in metre between the reverse curves shall not be less than 0.6 times the design speed in km/h.

b) up to 50% of the transition may be placed in the curve; and

c) where it is not possible to provide the minimum tangent distance in (a), the radius of each curve shall not be less than minimum radius of $V^2/127\,f$, where V is the design speed in km/h and f is the side friction factor.

10.4.2.3.4 Broken-back Curve

Broken-back curve consists of two curves with deflections in the same direction that is joined by a short tangent distance. In general, the use of broken-back curve is not favoured. Where it is necessary, the length of tangent in metre shall not be less than 3 times the design speed in km/h. Where such a minimum length of tangent cannot be obtained, the alignment shall be changed, either to increase the straight section, or eliminate it entirely by the use of a compound curve, or transition curve between the two curves.

10.4.2.3.5 Transition Curve

Transition curve is normally used to join a straight line to a circular curve, although it may be omitted when large radius curve is used. The most frequently used form of transition is the clothoid which curvature changes at a uniform rate along the curve. The following design requirements shall be adopted:

a) Where the design speed is up to 60km/h, no transition curve is required. Where the design speed is more than 60km/h, transition curve is not required if the associated shift in circular arc is less than 0.20m.

b) The length of the transition curve and shift can be derived from the following: -

$$L = \frac{V^3}{3.6^3 \, C \, R} \qquad p = \frac{L^2}{24 \, R}$$

Where,

C = Rate of change of radial acceleration (m/s^3)
for absolute minimum radius use $0.6 m/s^3$
for desirable radius use $0.3 m/s^3$

p = Shift (m)

L = Length of Transition (m)

R = Radius of circular curve (m)

V = Design Speed (km/h)

10.4.2.4 Crossfall of Carriageway

The crossfall of traffic lane and shoulder of straight section is provided to facilitate surface water drainage to the side drain and the design requirements shall be as shown in Table 10.6.

Table 10.6

	At-grade road and road on other structure	**Tunnel**
Desirable Crossfall	1 in 30	1 in 40
Absolute Minimum Crossfall	1 in 36	1 in 50

10.4.2.5 Super-elevation

10.4.2.5.1 The desirable super-elevation can be derived from the following equation:

$$e + f = \frac{V^2}{127 \, R}$$

Where,

e = Super-elevation (m/m)

f = Side friction factor (see Table 10.7)

V = Design Speed (km/h)

R = Radius of Curve (m)

Table 10.7

Design Speed, V (km/h)	**40**	**50**	**60**	**70**	**80**	**90**
Side Friction Factor, f	0.16	0.16	0.15	0.15	0.14	0.13

10.4.2.5.2 In cases where the negative or positive super-elevation derived is lower than the desirable crossfall, the desirable super-elevation shall be pegged at the crossfall as shown in Table 10.6 to slope towards the inner radius of the carriageway.

10.4.2.5.3 For rotation of pavement to attain super-elevation, the length required to develop the super-elevation shall satisfy the larger value obtained from the following two formulae:

(a) $L_e = \frac{|e_1 - e_2|V}{R}$ and (b) $L_e = |e_1 - e_2| W \times 100$

Where,

L_e = Super-elevation Development Length (m) (See Figure 10.5)

e_1 = Crossfall of carriageway (m/m) (see Table 10.6)

e_2 = Super-elevation at ends of the development length (m/m)

V = Design Speed (km/h)

W = Maximum width from axis of rotation to edge of running lane (m)

R = Rate of rotation;
use 0.126 for design speed less than 80 km/h
use 0.09 for design speed for 80 km/h & above

10.4.2.5.4 Where transition curve is provided, super-elevation shall be effected along the length of the curve. For simple circular curve, about two-third of the super-elevation development length shall be introduced on the tangent approach and one third on the curve.

10.4.2.5.5 The various percentages of super-elevation and the corresponding design speeds and radii are as shown in the Figure 10.6.

10.4.2.6 Corner Radius

The corner radius at an intersection affect the operation and safety of the intersection. The minimum radius shall not be less than the values as shown in Table 10.8.

Table 10.8

Location	Minimum Radius
Driveway to Primary / Local access	3 m
Driveway to Arterial Road	8 m
Other Road Junction	12 m
Industrial Estates	15 m

10.4.2.7 Taper Length

The length of the taper shall be in accordance with the latest LTA's Code of Practice for Traffic Control at Work Zone.

10.4.3 Vertical Alignment

10.4.3.1 Grade

10.4.3.1.1 Main Carriageway

The geometric design requirements of road shall be as shown in Table 10.9.

Table 10.9

Maximum Grade (%)	Road Category					
	1	**1A**	**2**	**3**	**4**	**5**
Desirable	4	5	5	6	6	6
Absolute	5	6	6	8	8	8

Notes:

1) Minimum gradient for all roads is 0.4%.
2) A road gradient of less than 0.4% may be used for widening of expressway.
3) For tunnel, minimum road gradient of 0.2% may be used.
4) For built-up area, a road gradient of less than 0.4% may be used to tie in with existing access.

10.4.3.1.2 Interchange Ramp/Loop and Slip Road

Table 10.10

Minimum Grade (%)	0.4
Desirable Maximum Grade (%)	6
Absolute Maximum Grade (%)	8

10.4.3.1.3 Critical Length of Grade

The length of grade shall be less than the critical values as shown in Table 10.11. Figure 10.7 illustrates the measurement of critical length of grade of a vertical curve.

Table 10.11

Grade (%)	**Critical Length of Grade (m)**
≤ 4	Unlimited
≤ 5	540
≤ 6	300
≤ 7	235
≤ 8	170

10.4.3.2 Types of Vertical Curve

A vertical curve shall be provided when there is a change in road gradient by using symmetrical simple parabolic curve.

10.4.3.2.1 Crest Curve

The minimum length of a crest curve in metre shall be the greater of:

a) 0.6 times of the design speed (in km/h); or

b) curve length needed to provide the minimum stopping sight distance (refer to Figure 10.8)

c) the desirable curve length can be derived from the following formulae: -

Crest Vertical Curve for Stopping Sight Distance

$$L = \frac{AS^2}{200\left(\sqrt{h_1} + \sqrt{h_2}\right)^2} \quad ; \qquad \text{when } L > S$$

$$L = 2S - \frac{200\left(\sqrt{h_1} + \sqrt{h_2}\right)^2}{A}; \qquad \text{when } L < S$$

Where,

A = Algebraic difference in road gradient (%)

h_1 = Height of eye above roadway surface(m), 1.15m

h_2 = Height of object above roadway surface (m), 0.2m

L = Length of vertical curve (m)

S = Stopping Sight Distance (m)

10.4.3.2.2 Sag Curve

The minimum length of a sag curve in metre shall be the greater of: -

a) 0.6 times of the design speed (in km/h); or

b) curve length needed to provide for riding comfort (refer to Figure 10.9); or

c) curve length needed to provide headlight sight distance (refer to Figure 10.10)

d) the desirable curve length can be derived from the following formulae: -

Sag curve for headlight sight distance

$$L = \frac{AS^2}{200\left(h + S(\tan q)\right)} \quad ; \qquad \text{when } L > S$$

$$L = 2S - \frac{200\left(h + S(\tan q)\right)}{A}; \qquad \text{when } L < S$$

Where,

A = Algebraic difference in road gradient (%)

h = Mounting height of headlight (m), 0.6m

L = Length of vertical curve (m)

q = Elevation angle of beam (upwards), 1 degree

S = Light beam distance (m) = Stopping Sight Distance (m)

10.5 ROAD CROSS-SECTIONS AND ELEMENT

10.5.1 General

The details of the cross-section elements such as lane width, centre median width, paved shoulder, services verge, footpath, drain and landscaping, etc. shall be in accordance with the Authority's Drawings and the Standard Details of Road Elements.

10.5.2 Lane Width

10.5.2.1 Main Carriageway

The typical lane width for a major road is 3.2m and the absolute minimum shall be 3m if there are constraints. In addition,

a) For lane adjacent to kerb, an additional width of 0.3m shall be provided.

b) (a) is not applicable if shoulder width indicated in Clause 10.5.4 are provided.

c) Consideration shall also be given to widen the lane further for swept path and other safety requirements on sharp horizontal curve.

The recommended desirable lane width shall be as shown in Table 10.12.

Table 10.12

Road Type	**Lane next to kerb**	**Other Lane**
Expressway	3.5m	3.5m[1]
Other Road (Two or more lanes)	3.5m	3.2m
Single-lane road	5.5m	-
Undivided 2-lane road	5.0m (For Industrial Area)	-
	3.7m (For Other Area)	-
Local Access	3.0m to 3.5m (depending on road reserve width and location)[2]	-

Notes:

1 Typical lane width for an expressway is 3.5m.

2 Actual lane width to be determined in consultation with Land Transport Authority

10.5.2.2 Ramp or Loop

The minimum lane width for ramp or loop is 3.7m per lane. The total width of the ramp or loop shall include the paved shoulder width specified in Clause 10.5.4. However, consideration shall be given to widen the lane width for the swept path of a heavy vehicle (refer to Figure 10.1).

10.5.2.3 Turning Lane

The turning lane at the intersection is to provide storage space for turning vehicle so as not to impede the main traffic flow (refer to Figure 10.11). The desirable minimum width of the turning lane shall be 3.5m and absolute minimum width shall be 3m. The turning lane shall be 70m long or sufficiently long to store the likely number of vehicles at any interval waiting to complete the turn, whichever is greater. A minimum of 30m taper is required for the transition.

To accommodate traffic lights, the desirable minimum and absolute minimum width of the centre divider at a junction separating the right turning lane from the opposing traffic flow shall be 1.3m and 1.0m respectively.

10.5.2.4 Slip Road

10.5.2.4.1 Slip road is provided to permit left-turning vehicle to bypass the intersection. The desirable merging angle for left turn slip road shall be 70^{o} and absolute minimum merging angle shall be 50^{o} as shown in Figure 10.12.

10.5.2.4.2 The minimum width of a single-lane slip road shall be 5.5m with a minimum turning radius of 30m to allow for the passing of a properly parked stalled vehicle. The minimum width for a two-lane slip road shall be 7.0m with a minimum turning radius of 25m.

10.5.2.4.3 For industrial area and slip road leading in/out of industrial area as shown in Urban Redevelopment Authority (URA) Conceptual Land Use Plan (refer to URA's website), the slip road width shall be increased to cater for the swept path of a heavy vehicle (refer to Figure 10.1). The design requirements for various radii of slip roads are shown in Table 10.13:

Table 10.13

Radius on Inner Edge of slip road (m)	**Width of one-lane slip road next to kerb (m)**		**Width of two-lane slip road next to kerb (m)**	
	Other areas	**Industrial area**	**Other areas**	**Industrial area**
15	6.0	7.8	7.8	9.4
25	6.0	6.9	7.0	8.6
30	5.5	6.7	7.0	8.4
50	5.5	6.3	7.0	7.9

Note:

Additional 0.3m lane width shall be added next to parapet/ barrier

10.5.3 Traffic Island

10.5.3.1 The desirable minimum dimension and the approach nose of the traffic island shall be as shown in the Figure 10.12.

10.5.3.2 Where there is a high concentration of pedestrian at the road intersection adjacent to school, shopping centre, hospital, food centre, the dimension of the traffic island shall be increased based on 0.3m^2 standing place per person. The number of pedestrians shall include those who are waiting to cross the road at the island before the start of the pedestrian crossing signal.

10.5.4 Paved Shoulder

10.5.4.1 The width of the paved shoulder shall be as shown in Table 10.14.

Table 10.14

<table>
<tr><th colspan="2">Road Type</th><th>Next to slow lane/ Left-hand side of carriageway</th><th>Next to fast lane/ Right-hand side of carriageway</th></tr>
<tr><td rowspan="2">Main carriageway</td><td>Expressway</td><td>2.75 m</td><td>0.9 m[1]</td></tr>
<tr><td>Other Road on structure[2]</td><td>2.0 m</td><td>0.6 m</td></tr>
<tr><td colspan="2">Ramp & Loop connecting expressway to expressway</td><td>2.0 m</td><td>0.6 m</td></tr>
</table>

Note:

[1] 1.0 m in tunnel

[2] Only applicable to viaduct, tunnel and associated depressed road. Paved shoulder is not required on flyover, underpass and associated depressed road.

10.5.4.2 For cases where paved shoulder cannot be provided, vehicle break down lay-by at 600m interval shall be provided and conform to the Standard Details of Road Elements.

10.5.5 Road Pavement

The types of flexible pavement for the various categories of road are shown in the Standard Details of Road Elements.

10.5.6 Sidetable, Drain, Footpath and Divider

The following shall be considered: -

a) Sidetable for drain and landscaping shall be provided outside the shoulder of the expressway or the carriageway of other categories of road. The services verge and landscaping shall be turfed and sloped as shown in the Standard Details of Road Elements.

b) For roads other than expressway or ramp and loop, unless otherwise specified, drain shall be slabbed over to be used as footpath. Where raised kerb is provided along the edge of carriageway, UPVC pipe with drop inlet chamber at 6m interval shall be provided to drain the surface water to the roadside drain. In addition, at the low point of a sag curve and at road junction, the spacing of the drop inlet chamber shall be at 3m interval for a distance of 30m measured between the points 15m from either sides of the lowest point.

c) Split level and open drain shall not be allowed at the divider.

d) Footpath shall be clear of any obstruction and footpath to adjoining properties shall be barrier-free in accordance with Building and Construction Authority (BCA)'s Code of Accessibility.

e) Footpath shall be higher than adjacent kerb as shown in the Standard Details of Road Elements. All footpaths exposed to weather shall be designed with falls and gradient to discharge water to the drains.

10.5.7 Vehicle Impact Guardrail

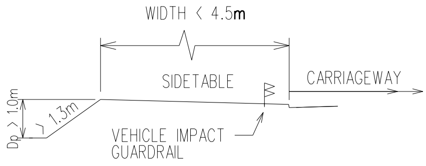
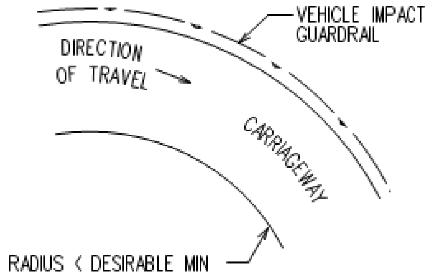
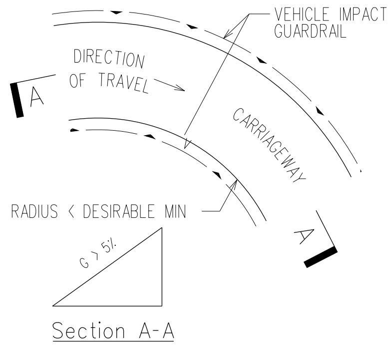
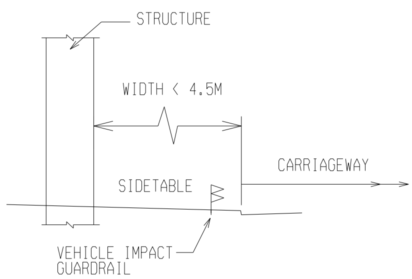
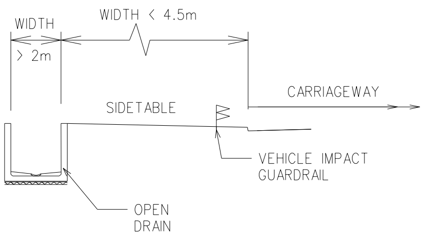
10.5.7.1 Vehicular impact guardrail shall be provided along expressway as shown in the Standard Details of Road Elements.

10.5.7.2 The arrangement of the vehicular impact guardrail shall be in accordance with the Standard Details of Road Elements. However, at specific location with site constraint, the placement of the vehicular impact guardrail shall take into consideration the necessary safety measures including the need to provide adequate visibility ahead for the driver of a vehicle.

10.5.7.3 In addition, the design shall also take into consideration the need to provide adequate protection where the safety of pedestrian and other facility are endangered.

10.5.7.4 For other at-grade road, vehicular impact guardrail shall be provided at the following locations as shown in Table 10.15.

Table 10.15

Location	Requirement	Illustration
(a) Slope	Where the top edge of the slope is less than 4.5m from outer edge of the carriageway and also coincide with the following conditions: (i) Difference in level between the top (carriageway level) and the toe of the slope is more than 1.0m; and (ii) Slope gradient (V:H) is greater than 1:3	
(b) Horizontal Curve	On the outer curve of carriageway where the radius is less than desirable minimum.	
(c) Grade	On both sides of the horizontal curves where the downhill road gradient is greater than 5%.	
(d) Bridge Approach and Other hazard	(i) At approach to bridge and parapet to refer to Standard Details of Road Elements; (ii) At other hazards such as bridge pier/ column and gantry sign support which are located less than 4.5m from the edge of carriageway.	
(e) Open drain	Internal width of drain exceeding 2.0m and located less than 4.5m from edge of carriageway.	

10.5.8 Emergency Gate

10.5.8.1 Emergency gate shall be provided for expressway (at-grade or viaduct) with consideration of each direction of traffic flow and the following requirements: -

a) an emergency gate shall not exceed 3km from an exit point, and

b) the interval between the successive emergency gates shall not exceed 3km.

10.5.8.2 The emergency gate shall be designed for crash-rated requirement complying with NCHRP Report 350 and BS EN 1317.

10.5.8.3 In addition, the design shall also take into consideration the sight distance requirement and other factors that may affect the safe functioning of the provision.

10.5.9 Bus Bay and Bus Stop without Bus Bay, Bus Shelter and Taxi Shelter and Passenger Pick-Up/Drop-Off Points

10.5.9.1 Bus bay, bus shelter, taxi stand and passenger drop-off points shall generally be provided at location close to where commuter can cross the road conveniently. The design of the bus bay shall be in accordance with the Standard Details of Road Elements. The design of bus shelter, taxi bay, taxi shelter and passenger drop-off points shall be in accordance with Architectural Design Criteria.

10.5.9.2 The following shall be taken into account during the provision of the bus bay, bus shelter, taxi stand and passenger drop-off points: -

a) it shall not be located along a curve unless sufficient sight distance can be provided;

b) it shall not be located right before the signalised pedestrian crossing unless overhead cantilever traffic signal pole can be provided;

c) it shall not be located right before non-signalised road junction unless sufficient intersection sight distance can be achieved;

d) outside a train station it shall be sited close to the entrance of the station;

e) along slip road, it shall be located not less than 100 m from the physical nosing;

f) it shall not be located within the storage lane at the junctions. There shall be a minimum offset of at least 25m from the end of the storage lane taper to the bus bay/ taxi stand/ passenger drop-off points;

g) there shall be a minimum distance of 30m between the bus bay/ taxi-stand/ passenger drop-off points and the traffic junctions; and

h) it shall have barrier-free access without steps in accordance with BCA's Code of Accessibility.

Notes applicable only for Bus Bay & Bus Shelter

i) the interval between bus bays varies from 350m to 400m for major arterial road; and 300m to 350m in heavily built-up area and new town;

j) the siting of a pair of bus bays and shelters shall be provided as shown in Figure 10.13; and

k) near signalised road junction, bus bay and bus shelter shall be sited as close to the junction as possible to provide convenience for the commuters who might be transferring to a bus on the opposite carriageway. Figure 10.14 and Figure 10.14.1-10.14.4 refer to the minimum distance required between the signalised junction and the bus bay

10.5.10 Motorcyclist Rainshelter

A separate space for a rainshelter shall be provided for motorcyclist away from the traffic lane of expressway. Reference shall be in accordance with Standard Details of Road Elements and Table 10.16.

Table 10.16

Provision	**Expressway**	
	At-Grade	**Tunnel**
Motorcyclist Rainshelter	Along the expressway directly underneath flyover or viaduct which overcrosses the expressway	Along the expressway at the exit of the tunnel box

10.5.11 Road Signage

10.5.11.1 The design of the road sign shall be in accordance with the Standard Details of Road Elements and M&W Specification.

10.5.11.2 Road sign shall be suitably located to enable driver to have sufficient time to react safely.

10.5.11.3 In general, road signs are classified as follows:

a) Regulatory Sign: This imposes legal restriction

b) Warning sign: This calls attention to hazardous condition

c) Information/ Directional sign

10.5.11.4 Directional Sign comprises the following:

a) Advance and Intermediate Directional Sign is to inform road user of the routes ahead before reaching an interchange or road intersection.

b) Confirmatory Directional Sign is to give the route information at an interchange or road intersection.

10.5.11.5 Directional sign shall generally be sited on the side next to the slow lane and clearly visible to the driver. It shall be noted that in some cases (e.g. for a major right turning movement, or for very wide road) it may be appropriate to site sign on the median if it is wide enough. The minimum lateral clearance from the edge of the vehicular impact guardrail to the edge of the signboard shall be at least 600mm. The post/support of the signboard shall be at least 1000mm from the vehicular impact guardrail. Where there is no vehicular impact guardrail next to the raise kerb, the minimum clearance between the edge of signboard and the edge of road kerb shall be at least 600mm.

10.5.11.6 The siting of the directional sign shall be in accordance with Standard Details of Road Elements.

10.5.11.7 The standard directional sign specified in the Standard Details of Road Elements shall be used for at-grade road and grade-separated interchange (i.e. underpass, flyover, tunnel, viaduct) provided the visibility and the geometric design requirements meet the design standard in the Design Criteria.

10.5.11.8 For grade-separated interchange on other road categories where the visibility and geometric design requirements cannot be complied with the Design Criteria due to site constraint, then enhanced guidance (traffic sign showing both destination and lane use) shall be provided either by using an overhead gantry sign, cantilever sign or lane indication sign (as shown in Figure 10.15). The option to select any of these signs shall depend on the site condition and is subject to the Authority's approval.

10.5.11.9 Road sign shall be sited at 95^{o} away from the line of a straight carriageway to avoid direct reflection from headlamp beam. For left hand bend, the 95^{o} angle shall be measured from a line joining the sign to a point at the edge of the carriageway and 180m before the sign as shown in Figure 10.16.

10.5.11.10 All signs shall not be blocked by trees. On expressway within 75m in front of a Directional Sign, no tree shall be planted. On other roads, the corresponding distance is 45m.

10.5.12 Interchange

10.5.12.1 Ramp

10.5.12.1.1 Entrance and exit ramps of an interchange shall be designed for safe and efficient operation. They shall be located on tangent section in order to provide maximum sight distance.

10.5.12.1.2 For major arterial road and semi-expressway, the details of the acceleration or deceleration lane shall be as shown in Figure 10.17 under the conditions given in Figure 10.18.

10.5.12.1.3 The distance between successive entrance or exit ramp shall be taken into consideration for the installation of directional signs.

10.5.12.2 Gore

10.5.12.2.1 The gore is an interchange or intersection as shown in Figure 10.19.

10.5.12.2.2 The gore nose (i.e. start of the gore) is typically 6m wide measured between the travelled way of the two roads (including the paved shoulder) and is about 20m downstream from the start of physical nosing. The width of the gore nose may be increased if the ramp curves away from the main carriageway.

10.5.12.2.3 Where the provision of 20m x 6m clear zone cannot be achieved, an appropriate crash-rated system complying with NCHRP Report 350 and BS EN 1317 and any subsequent revisions shall be provided with respect to the design speed of the road. Reference shall be in accordance with Standard Details of Road Elements.

10.5.12.2.4 The following shall be considered when designing the gore of an exit ramp: -

a) the neutral area shall be graded as nearly level with the road and shall also be free of tree, lamp post, sign, other physical obstacles etc. except split arrow and object marker sign. The drain at the neutral area shall also be covered with grating;

b) the gore shall be graded as flat as practicable;

c) where the placement of a major obstruction, such as a bridge pier in a gore is unavoidable, an appropriate treatment shall be installed in front to reduce the severity of fixed object collision. The appropriate treatment shall be located on a level area free from kerbs and other physical obstacles;

d) there shall be proper advance signs for the exit and clear road marking at the gore area;

e) the maximum algebraic difference between the crossfalls of the through lane of the main carriageway and the exit ramp when they are adjacent to each other shall not exceed 4%; and

f) the first lamp pole will be installed about 500mm after the neutral area.

10.6 DRAINAGE DESIGN

The drainage design shall be in accordance with PUB's Code of Practice on Surface Water Drainage.

10.7 CLEARANCE TO STRUCTURES

No structure shall be erected above the carriageway or too near to the carriageway that they become hazard to the road users. The minimum allowable lateral and vertical clearance shall satisfy the following clauses.

10.7.1 Lateral Clearance

10.7.1.1 Minimum lateral clearance from the edge of road pavement (both at centre median and side table) to any adjacent structure is shown in Table 10.17 unless otherwise specified in the Standard Details of Road Elements.

Table 10.17

Types of Structure	Expressway	Other Road
Commuter facility structure (such as landing, staircase, column of Pedestrian Overhead Bridge, column of covered linkway, lift shaft, underpass entrance/exit), flyover/viaduct column, other above ground structures (such as MRT station entrance, vent shaft, exit staircase, emergency escape staircase, etc.) and support of overhead gantry.	2.0 m	1.2 m
Abutment walls of Flyover	3.0 m	3.0 m
Over ground boxes, lamp post, controller box, traffic sign post	1.8m	0.6 m[1] 1.2 m[2]

Notes:

[1] If no Vehicular Impact Guardrail is installed. Applicable to both centre median and sidetable.

[2] Where Vehicular Impact Guardrail is installed.

10.7.1.2 For an elevated road structure, the minimum lateral clearance between the edge of the road structure and any other structures shall be 3m.

10.7.1.3 For vehicular tunnel/underpass/depressed road, the minimum lateral clearance between the edge of the carriageway and the inner face of the wall shall be as shown in Figure 10.20.

10.7.1.4 The above requirements on lateral clearance are not intended to constitute a clear zone. Further additional clearances and protection devices where appropriate have to be provided with consideration to safety, operational and maintenance needs, etc.

10.7.2 Vertical Clearance

The minimum vertical clearance from the road surface to the soffit of structures shall be as specified in Table 10.18.

Table 10.18

Types of Structure	All Roads
Pedestrian Overhead Bridge, Flyover, Rail Viaduct	5.4 m
Overhead Gantry Signs	5.7 m
Soffit of deck of road viaduct running parallel to and above the road	10 m

10.8 ROAD TUNNEL

10.8.1 For vehicular tunnel, a minimum structure clearance of 5.9m is required as shown in Figure 10.20 with a clear envelope of 5.1m.

10.8.2 For vehicular underpass, a minimum structural clearance of 5.6m is required as shown in Figure 10.20 with a clear envelope of 5.1 m.

10.8.3 Collision Protection Beam

10.8.3.1 One Collision Protection Beam shall be installed prior to entry of a vehicular tunnel. As far as possible the Collision Protection Beam shall be integrated with bridge supports or tunnel structures. Where there is site constraints, a stand-alone collision protection gantry which is designed according to the standard shall be provided so long as its intrusiveness is acceptable.

10.8.3.2 Height Restriction Gantry where it functions as an advanced physical warning device to alert overhead vehicles about the approach to the tunnel shall be installed before the Collision Protection Beam, at suitable locations where over-height vehicles can safely manoeuvre out to other roads. The lowest point of the gantry frames shall be at least 0.3m below the overhead structures or any object above the carriageway. Reference shall be made with Standard Details of Road Elements.

10.8.4 Cross Passage Openings and Escape Staircases

10.8.4.1 For road tunnel and underground slip roads greater than 100m, the maximum interval for cross passages and escape emergency staircases shall comply with the Singapore Civil Defence Force's latest requirements, NFPA 502 and other relevant codes.

10.8.4.2 No steps are to be provided at cross passage between the different bounds of carriageway, unless it is not technically feasible to do so.

10.8.4.3 In addition to the above requirements, the design shall also take into consideration the sight distance requirements and other factors that may affect the safe functioning of the provision.

10.8.4.4 For main tunnel, cross passage for vehicles shall be provided at maximum intervals of 1,500m

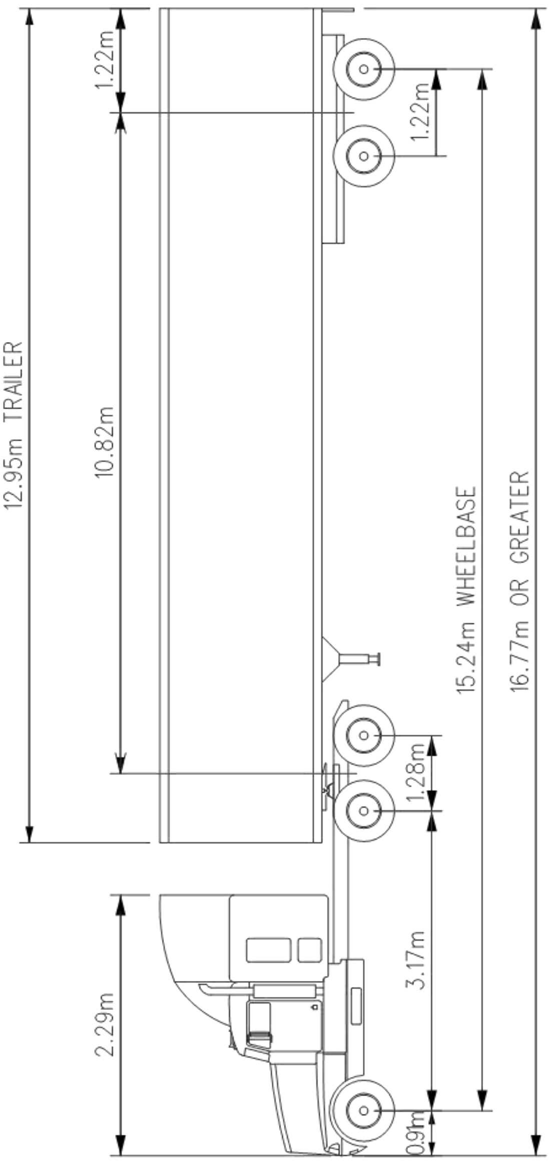


Figure 10.1
Heavy Vehicle

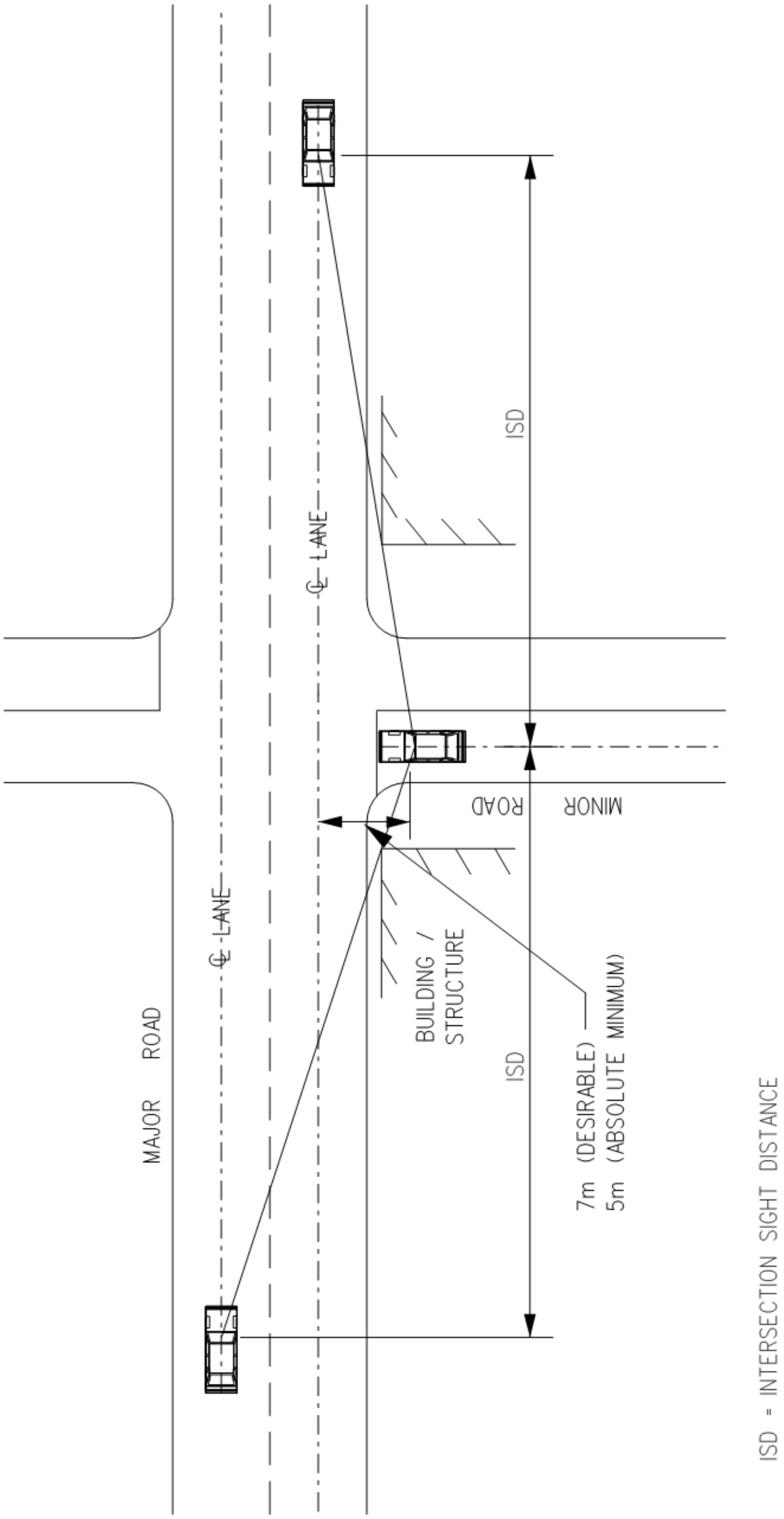


Figure 10.2
Sight Triangle

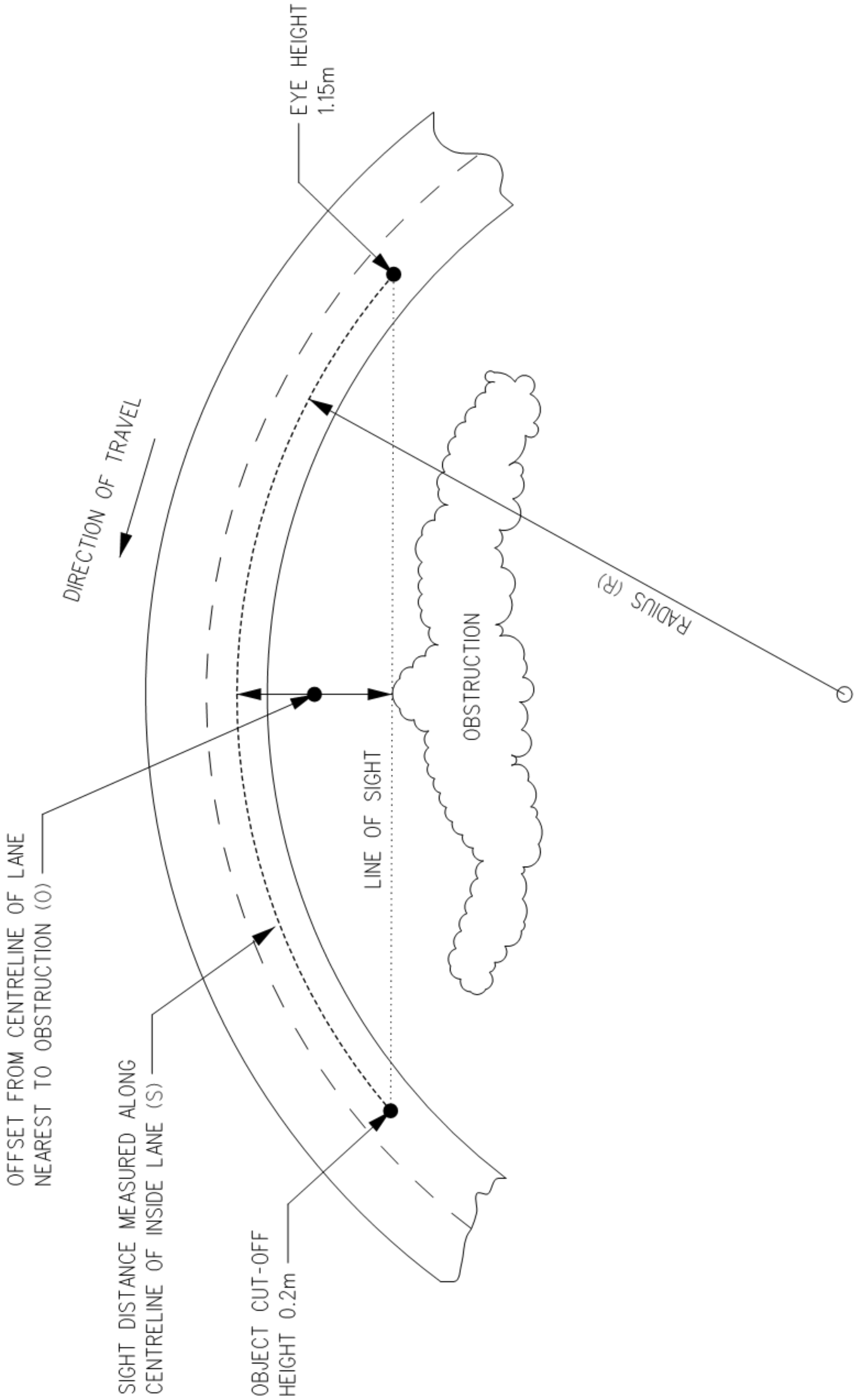


Figure 10.3
Visibility Around Horizontal Curve

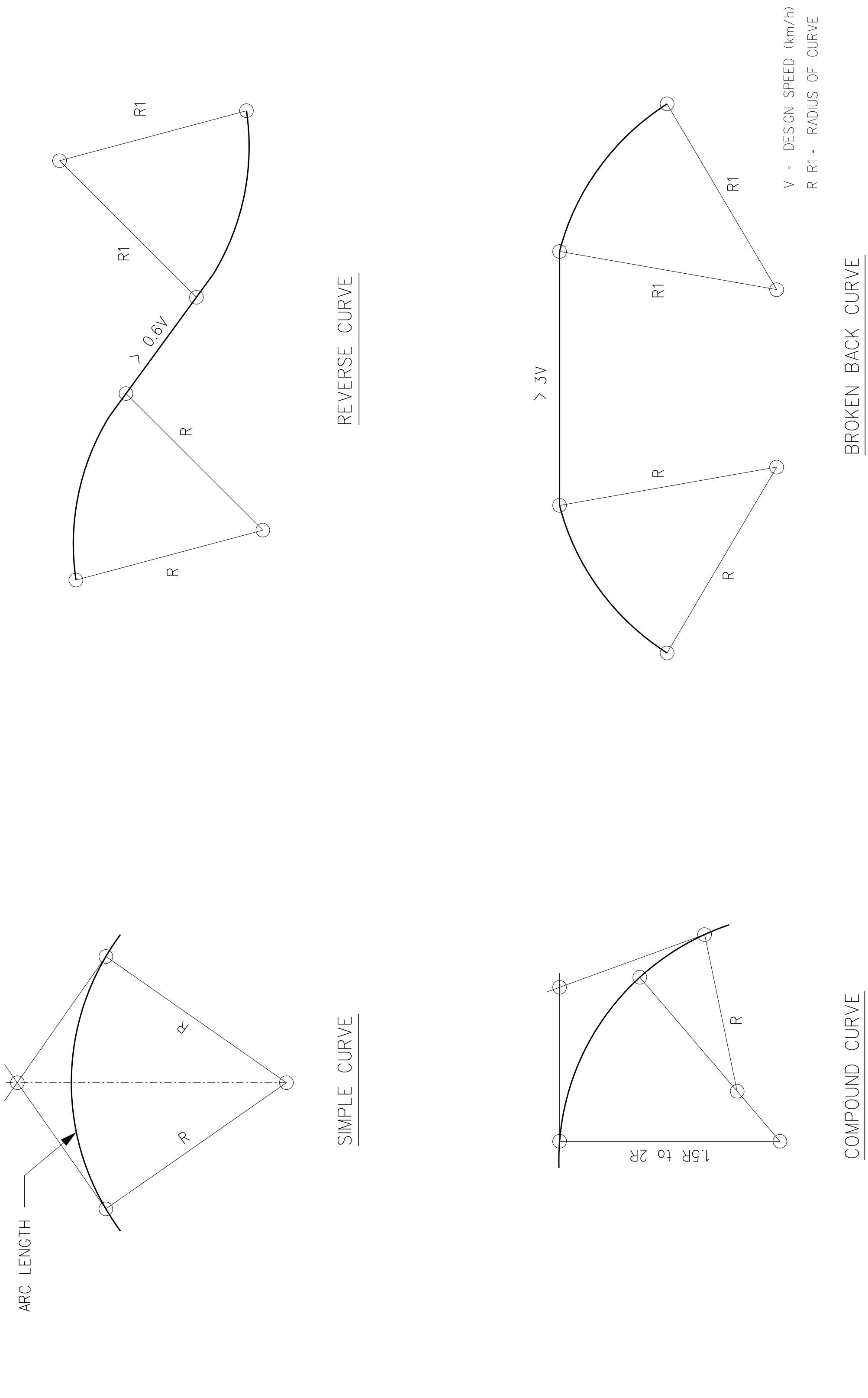
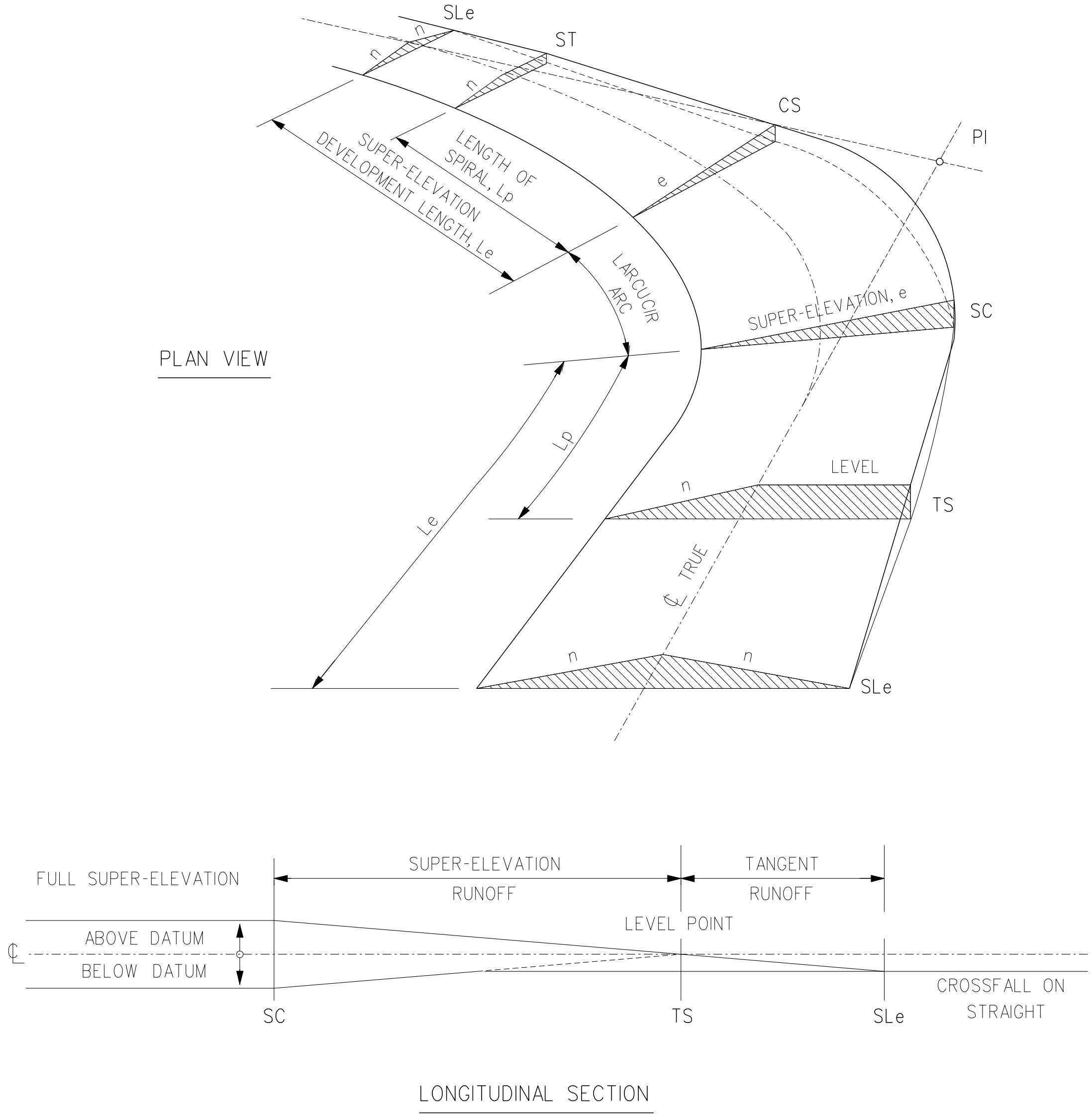


Figure 10.4
Types of Curves



PI	POINT OF INTERSECTION OF THE MAIN TANGENTS
TS	TANGENT TO SPIRAL - COMMON POINT OF TANGENT AND SPIRAL
SLe	START OF SUPER-ELEVATION DEVELOPMENT
SC	SPIRAL TO CURVE - COMMON POINT OF SPIRAL AND CIRCULAR CURVE
Lp	LENGTH OF SPIRAL, TS TO SC (m)
Le	LENGTH OF SUPER-ELEVATION DEVELOPMENT (m)
n	NORMAL PAVEMENT CROSSFALL (%)
e	PAVEMENT SUPER-ELEVATION (%)

Figure 10.5

<u>Development of Super-elevation</u>

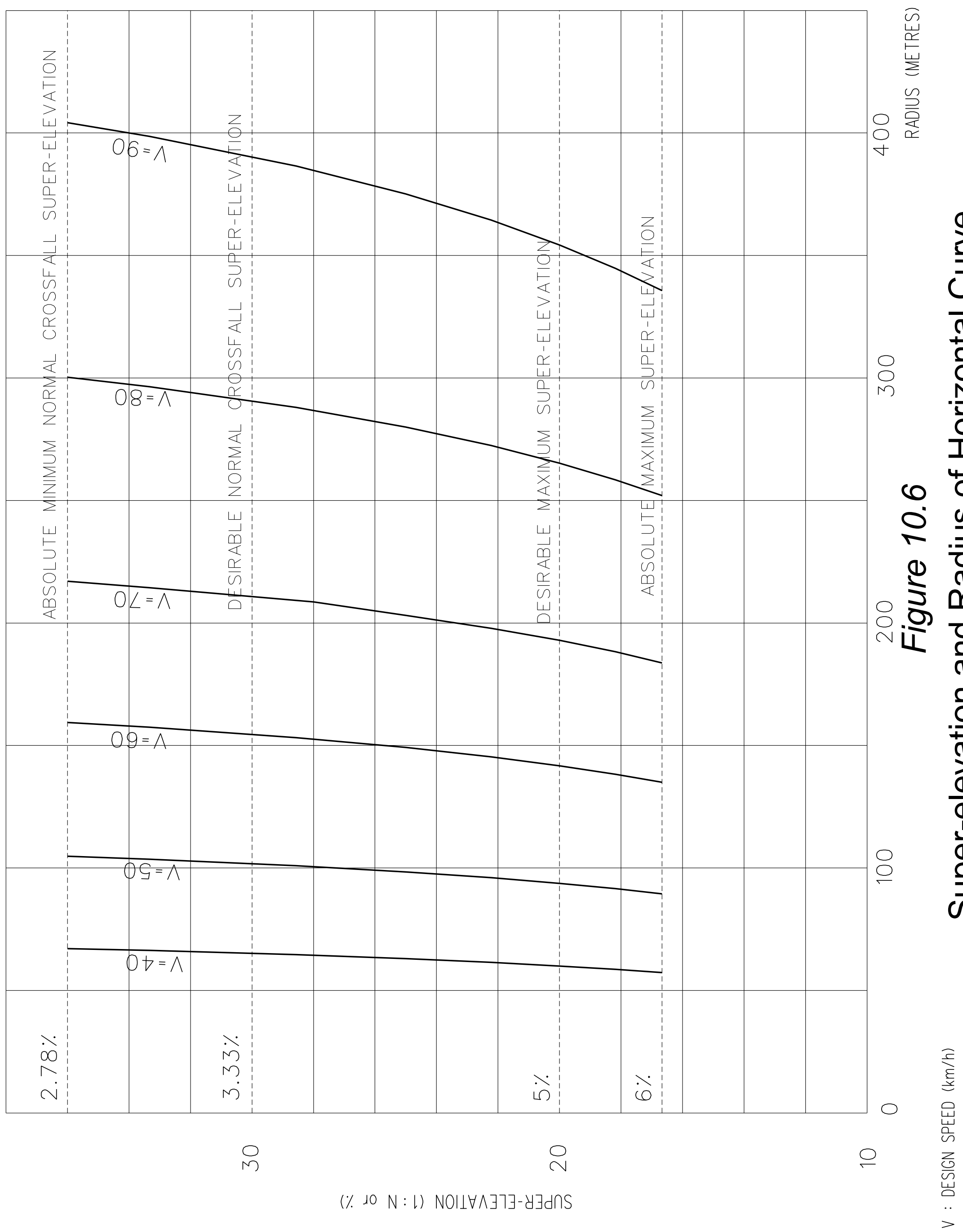
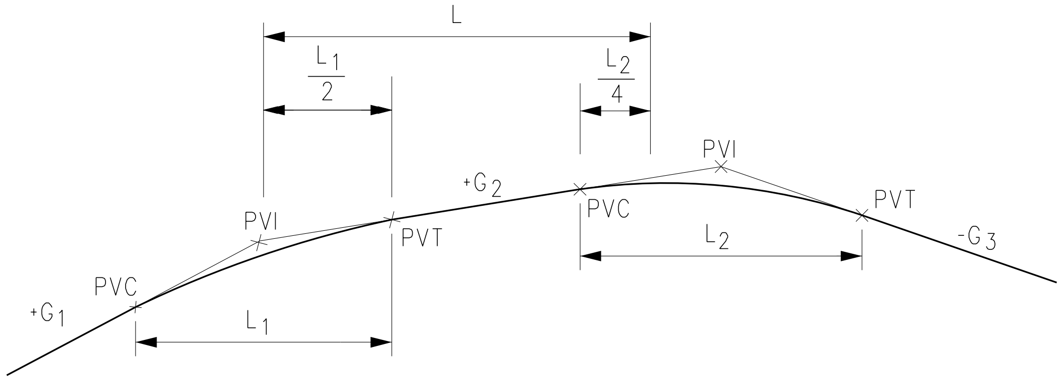


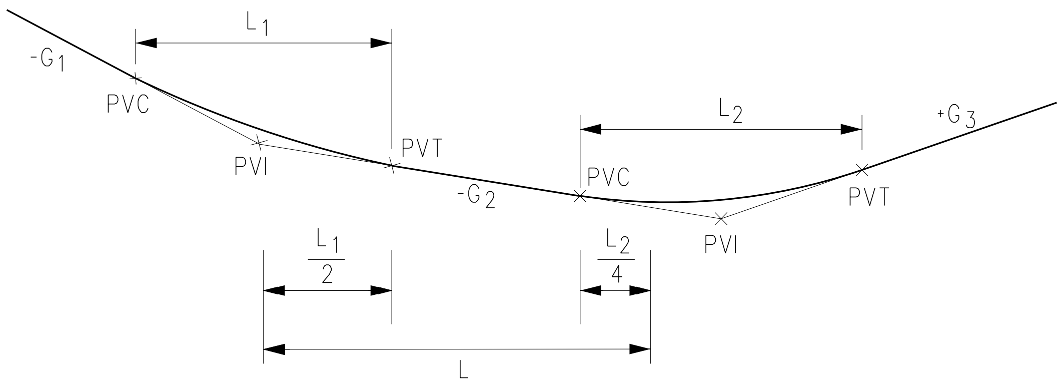
Figure 10.6

Super-elevation and Radius of Horizontal Curve

SCALE 1 : 200 (Vertical)
SCALE 1 : 2500 (Horizontal)



CREST VERTICAL CURVE



SAG VERTICAL CURVE

LEGEND

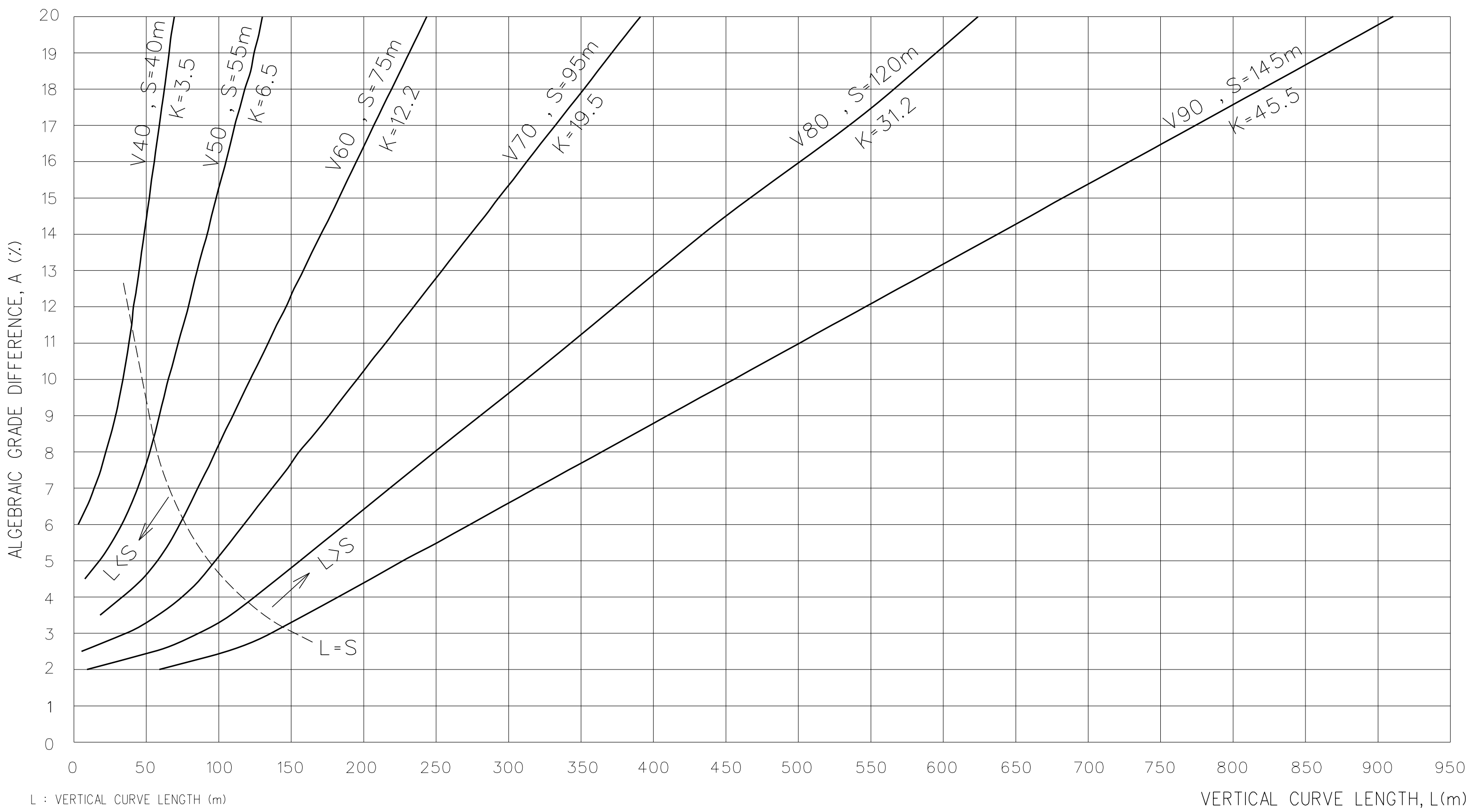
L : CRITICAL LENGTH OF GRADE
L_1 L_2 : VERTICAL CURVE LENGTH
PVC : POINT OF VERTICAL CURVE
PVI : POINT OF VERTICAL INTERSECTION
PVT : POINT OF VERTICAL TANGENT
G : GRADIENT

NOTES :

1. FOR VERTICAL CURVES WHERE THE TWO TANGENT GRADES ARE IN THE SAME DIRECTION (BOTH UPGRADES OR BOTH DOWNGRADES), 50% OF THE CURVE LENGTH WILL BE PART OF THE LENGTH OF GRADES.
2. FOR VERTICAL CURVES WHERE THE TWO TANGENT GRADES ARE IN THE OPPOSITE DIRECTIONS (ONE GRADE UP AND ONE GRADE DOWN), 25% OF THE CURVE LENGTH WILL BE PART OF THE LENGTH OF GRADES.
3. THE ABOVE DIAGRAM IS INCLUDED FOR ILLUSTRATIVE PURPOSE ONLY. BROKEN BACK CURVES ARE TO BE AVOIDED WHENEVER PRACTICAL.

Figure 10.7

Measurement for Critical Length of Grade



L : VERTICAL CURVE LENGTH (m)
V : DESIGN SPEED (km/h)
S : STOPPING SIGHT DISTANCE (m)
K : L/A
A : ALGEBRAIC GRADE DIFFERENCE (%)

Notes :

1) The following table is to be used for levelroad or where road gradient is less than 2%.
2) For road gradient more than 2% (up-grade or down-grade), the information can be derived from the formula.

Figure 10.8

Crest Vertical Curve Length for Stopping Sight Distance

SCALE 1 : 200 (Vertical)
SCALE 1 : 5000(Horizontal)

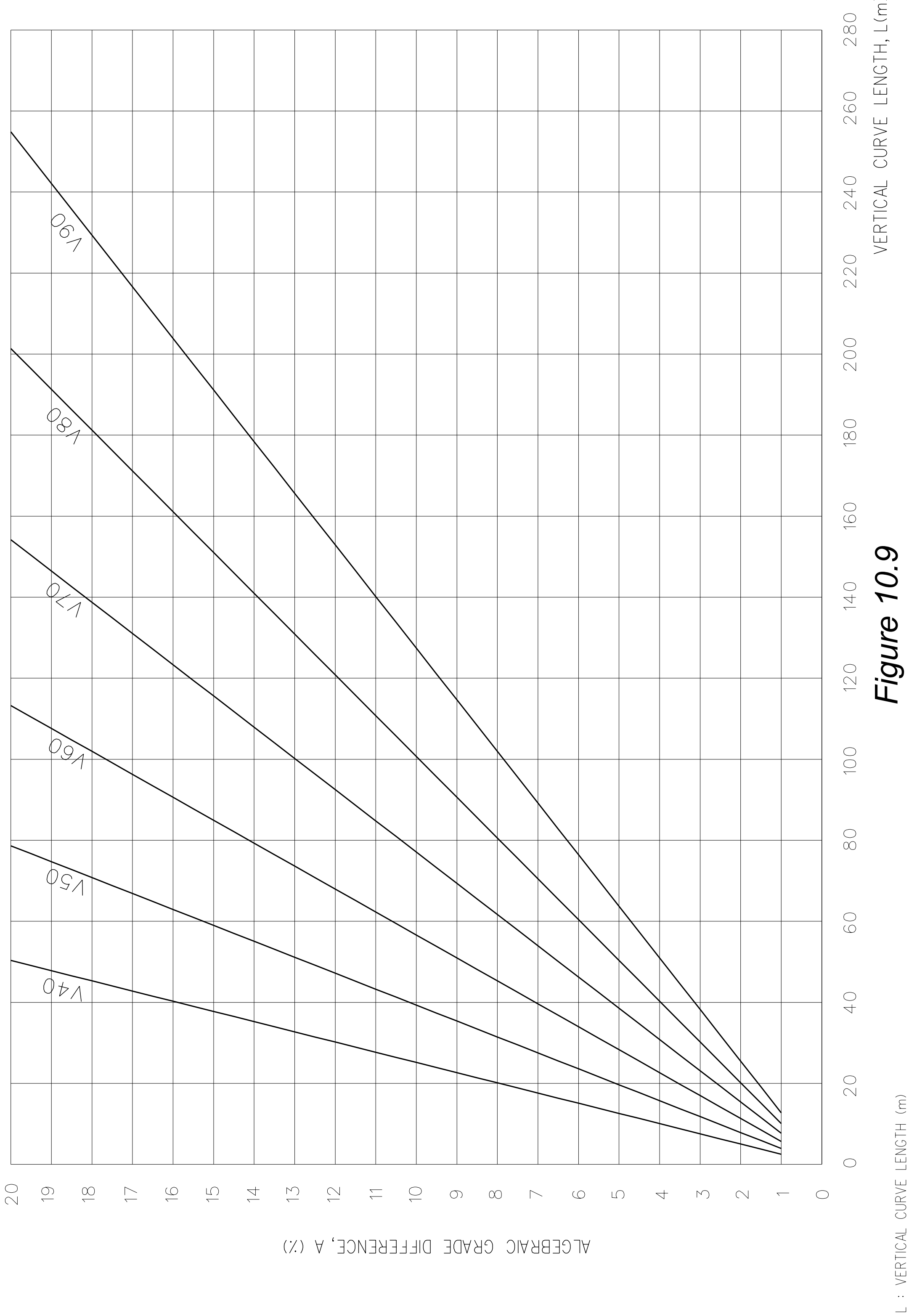
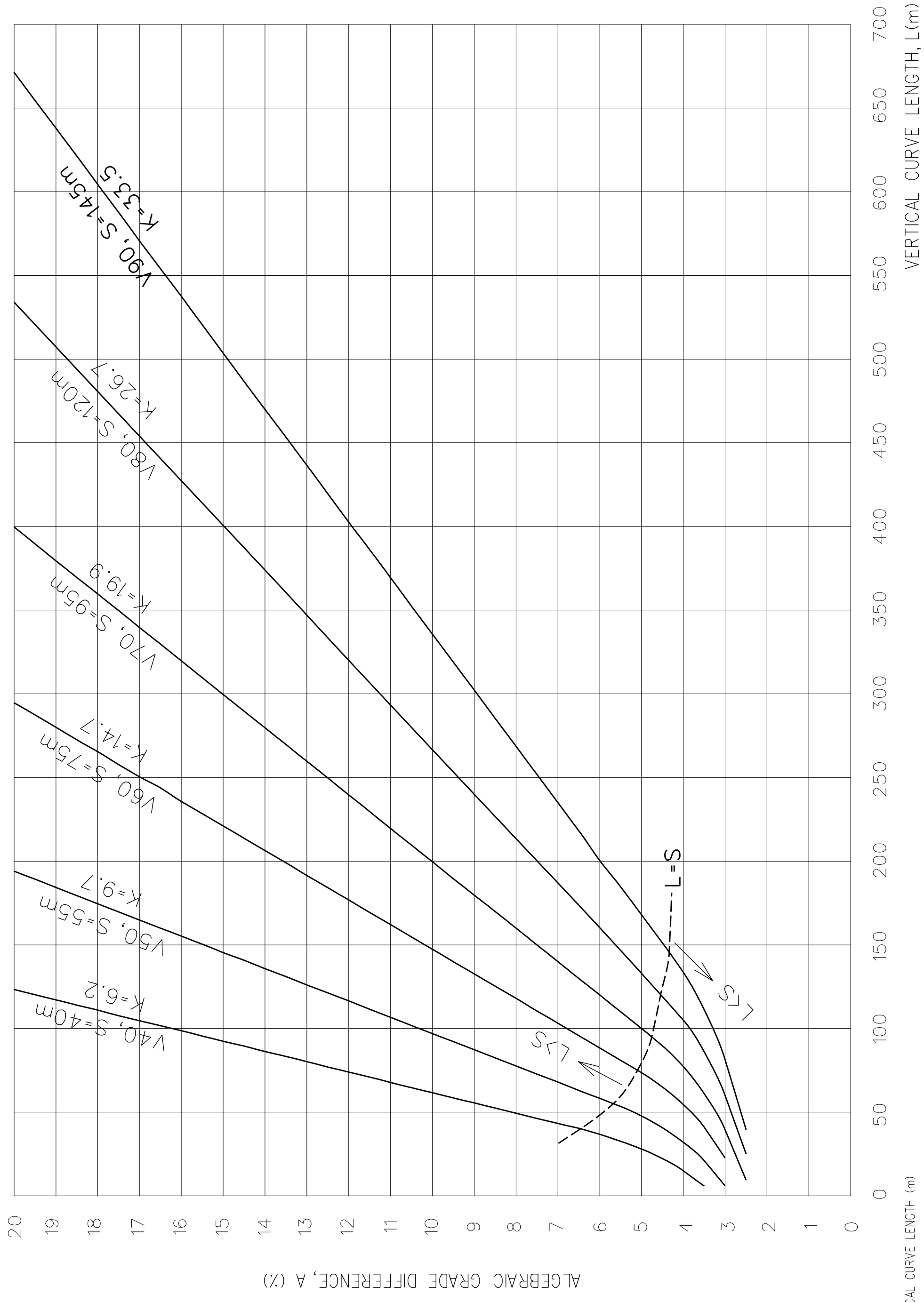


Figure 10.9

Sag Vertical Curve Length for Riding Comfort

SCALE 1 : 150 (Vertical)
SCALE 1 : 1500 (Horizontal)



L : VERTICAL CURVE LENGTH (m)
V : DESIGN SPEED (km/h)
S : STOPPING SIGHT DISTANCE (m)
K : L/A
A : ALGEBRAIC GRADE DIFFERENCE (%)

Notes :

1) The following table is to be used for levelroad or where road gradient less than 2%.
2) For road gradient more than 2% (up-grade or down-grade), the information can be derived from the formula.

Figure 10.10

Sag Vertical Curve Length for Headlight Sight Distance

SCALE 1 : 150 (Vertical)
SCALE 1 : 3750 (Horizontal)

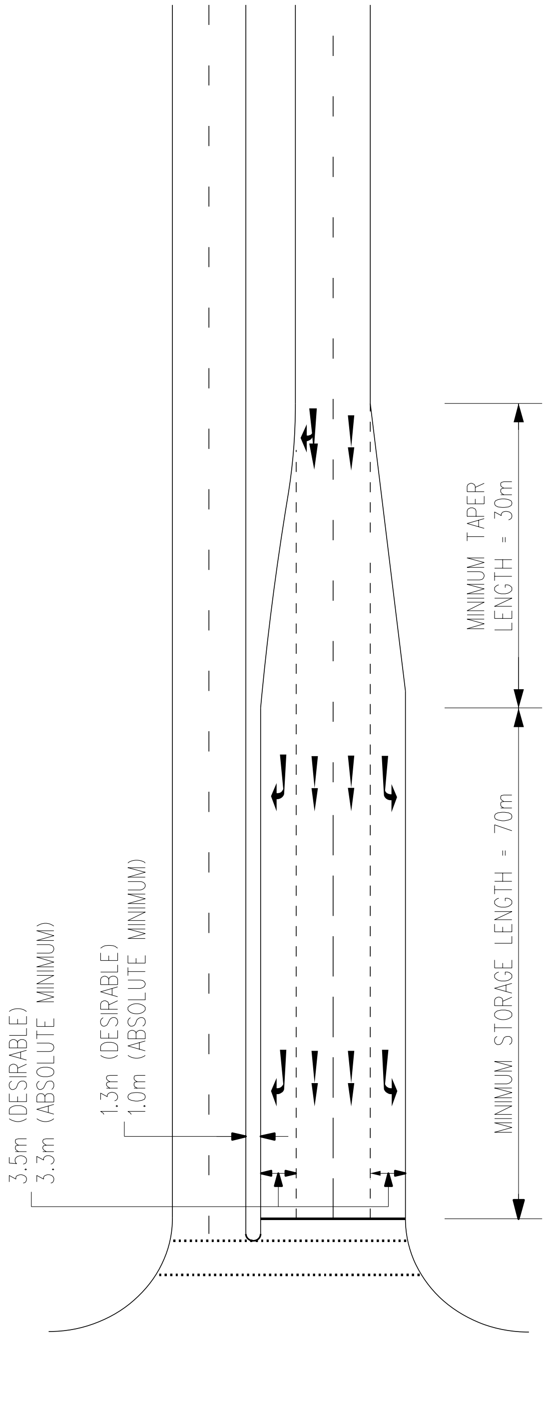


Figure 10.11

Turning Lane Layout

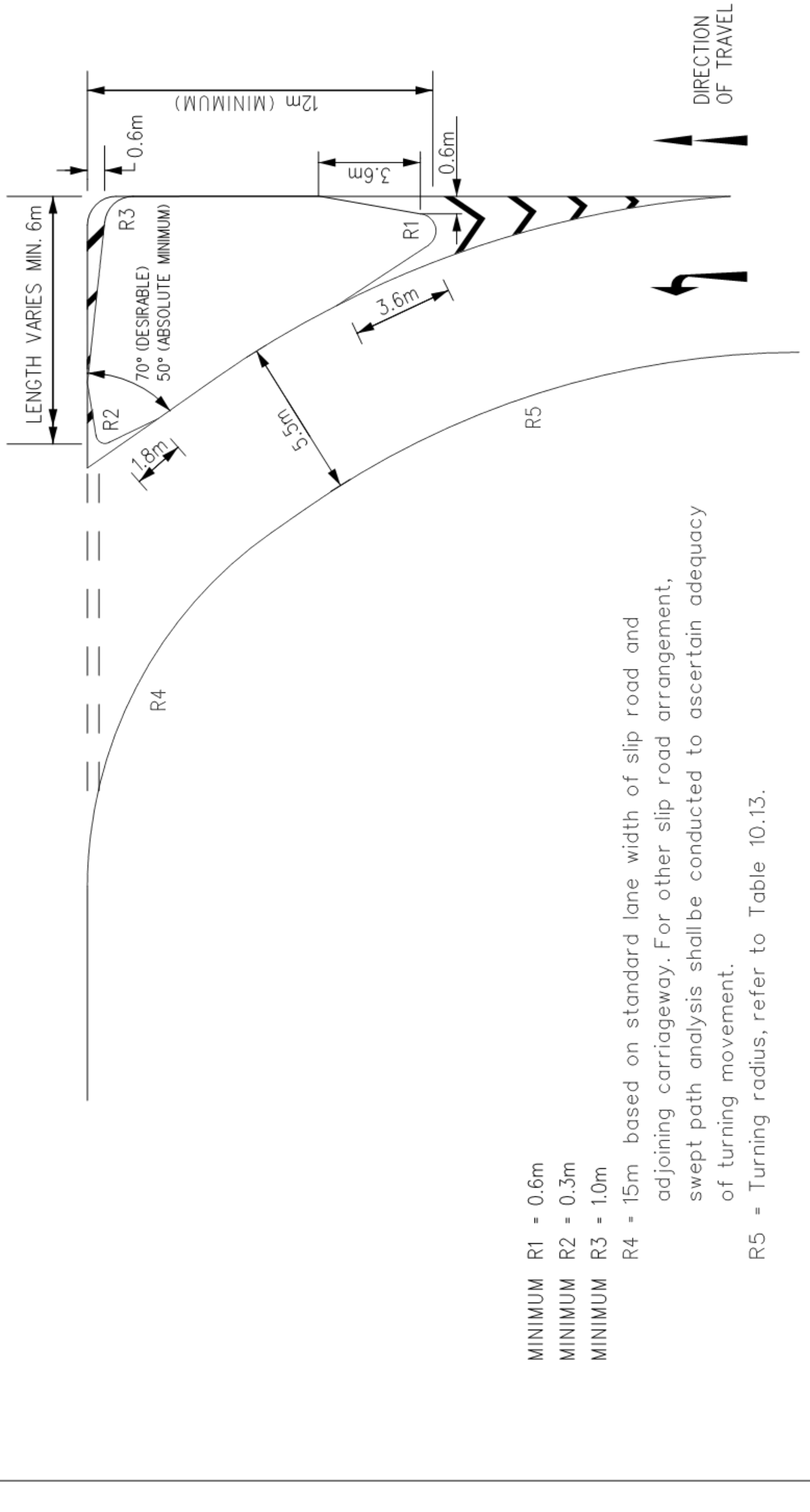


Figure 10.12
Desirable Minimum Traffic Island

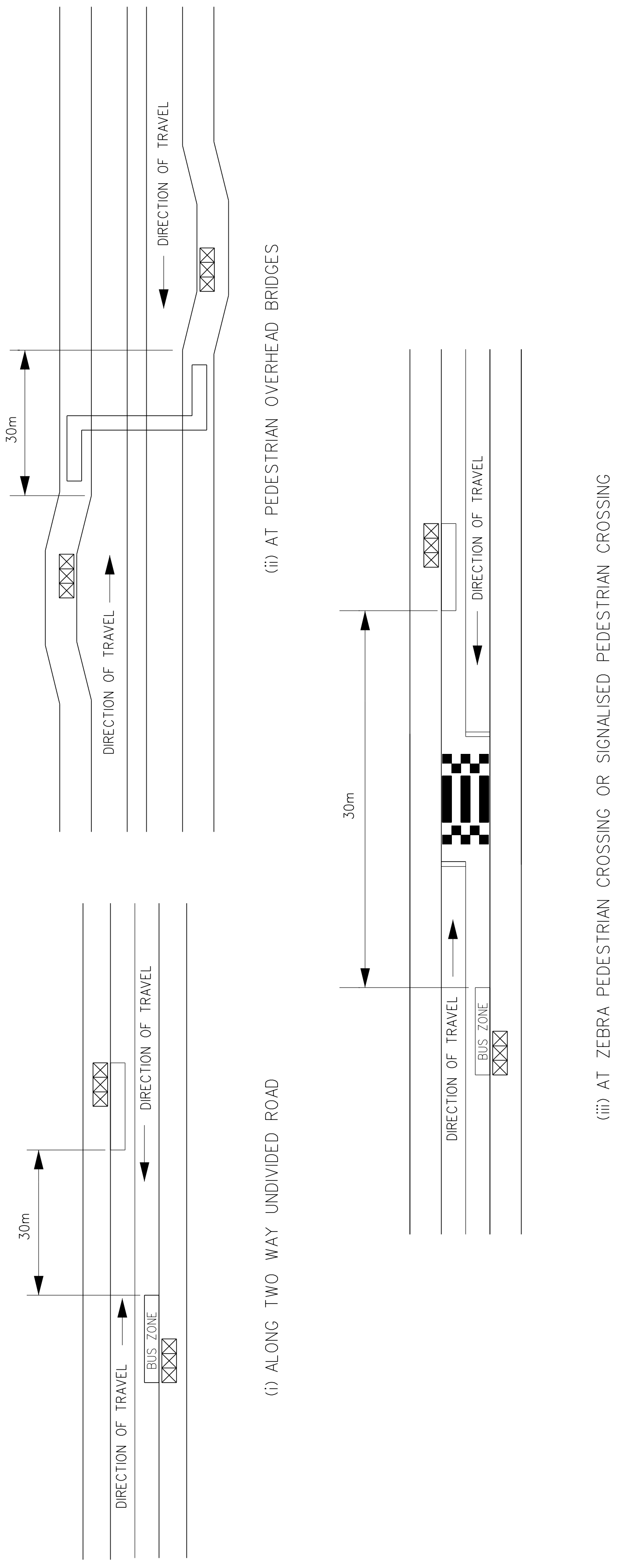


Figure 10.13

Siting of a Pair of Bus Bays and Shelters

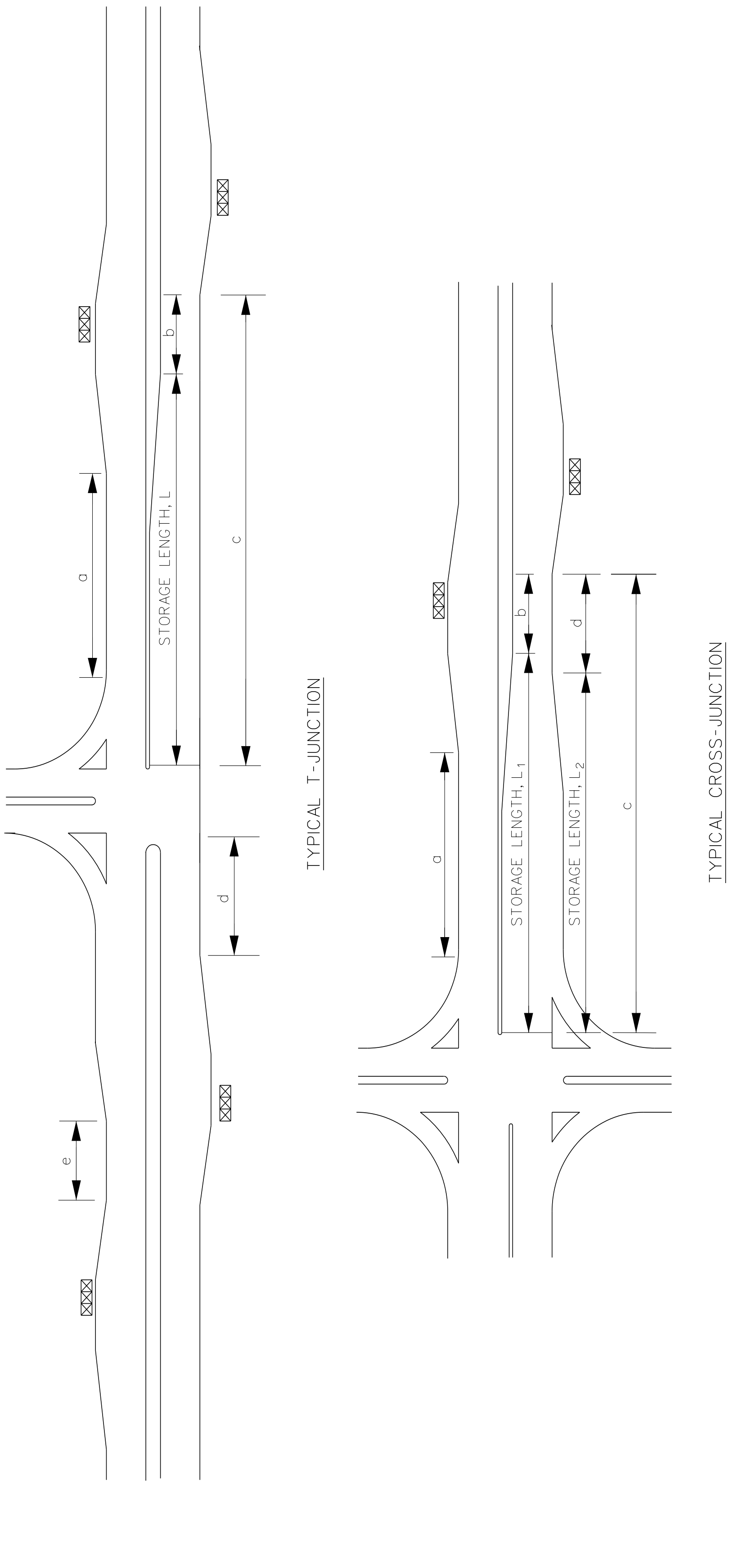


Figure 10.14
<u>Siting of Bus Bay and Shelter</u>

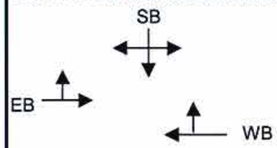
Figure 10.14.1

GUIDELINE FOR SITING OF BUS BAY ADJACENT TO T-JUNCTION

A typical T-junction layout and the denotation indicated in the table below is shown in the Figure 10.14.

Assumptions made for this guideline :

1. The road is a divided carriageway.
2. No site constraint.

Scenario	Minimum Dimension in metre					Remarks
	a [1]	b	c [2]	d [1]	e	
Case 1 : Bus Routes in all directions : 						
1) Bus bay provided :						
a) weave thru 2 lanes	30	25	80	30	20	Yellow box across the carriageway to be provided. See Note 3.
b) weave thru 3 lanes	30	25	120	30	20	Yellow box across min. 1-lane of the carriageway to be provided. See Note 3.
c) weave thru 4 lanes	30	25	160	30	20	Yellow box across min. 2-lanes of the carriageway to be provided. See Note 3.
2) Without bus bay :						
a) weave thru 2 lanes	50	25	40	50	20	See Note 3.
b) weave thru 3 lanes	50	25	80	50	20	See Note 3.
c) weave thru 4 lanes	50	25	120	50	20	See Note 3.

Note:

[1] If the bus volume at the bus bay is high (exceed 120 buses/hr), double tier or higher capacity bus bay is required.
LTA's Bus & Taxi Regulation Dept and Traffic Management will need to be consulted for dimension of a & d.

[2] For dimension of c, it refers to the minimum weaving distance for right turning buses.

[3] In the Westbound (WB) direction, the location of bus bay before the junction should be c or (L+b) whichever is greater.

Figure 10.14.2

GUIDELINE FOR SITING OF BUS BAY ADJACENT TO T-JUNCTION

A typical T-junction layout and the denotation indicated in the table below is shown in the Figure 10.14.

Assumptions made for this guideline :

1. The road is a divided carriageway.
2. No site constraint.

Scenario	Minimum Dimension in metre					Remarks
	a [1]	b	c	d [1]	e	
Case 2 : Bus Routes with no right turning at junction : SB EB WB						
1) Bus bay provided :						
a) weave thru 2 lanes	30	0	30	30	20	Yellow box across 1-lane of the carriageway to be provided. See Note 2.
b) weave thru 3 lanes	30	0	30	30	20	Yellow box across 1-lane of the carriageway to be provided. See Note 2.
c) weave thru 4 lanes	30	0	30	30	20	Yellow box across 1-lane of the carriageway to be provided. See Note 2.
2) Without bus bay :						
a) weave thru 2 lanes	50	0	40	50	20	See Note 2.
b) weave thru 3 lanes	50	0	40	50	20	See Note 2.
c) weave thru 4 lanes	50	0	40	50	20	See Note 2.

Note:

[1] If the bus volume at the bus bay is high (exceed 120 buses/hr), double tier or higher capacity bus bay is required.
LTA's Bus & Taxi Regulation Dept and Traffic Management will need to be consulted for dimension of a & d.

[2] LTA's Bus & Taxi Regulation Dept will need to be consulted and the Authority must agreed that there is no future bus route turning right in the Westbound direction.

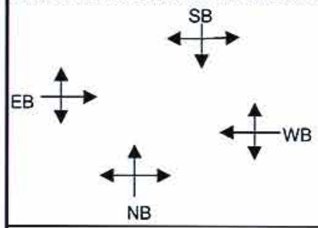
Figure 10.14.3

GUIDELINE FOR SITING OF BUS BAY ADJACENT TO CROSS-JUNCTION

A typical Cross-junction layout and the denotion indicated in the table below is shown in the Figure 10.14.

Assumptions made for this guideline :

1. The road is a divided carriageway.
2. No site constraint.

Scenario	Minimum Dimension in metre				Remarks
	a[1]	b	c[2]	d	
Case 1 : Bus Routes in all directions : 					
1) Bus bay provided :					
a) weave thru 2 lanes	30	25	80	20	Yellow box across the carriageway to be provided. See Note 3.
b) weave thru 3 lanes	30	25	120	20	Yellow box across min. 1-lane of the carriageway to be provided. See Note 3.
c) weave thru 4 lanes	30	25	160	20	Yellow box across min. 2-lanes of the carriageway to be provided. See Note 3.
2) Without bus bay :					
a) weave thru 2 lanes	50	25	40	20	See Note 3.
b) weave thru 3 lanes	50	25	80	20	
c) weave thru 4 lanes	50	25	120	20	See Note 3.

Note:

[1] If the bus volume at the bus bay is high (exceed 120 buses/hr), double tier or higher capacity bus bay is required.
LTA's Bus & Taxi Regulation Dept and Traffic Management will need to be consulted for dimension of a & d.

[2] For dimension of c, it refers to the minimum weaving distance for right turning buses.

[3] In the Westbound (WB) direction, the location of bus-stop before the junction should be c or (L_1+b) or (L_2+d) whichever is greater.

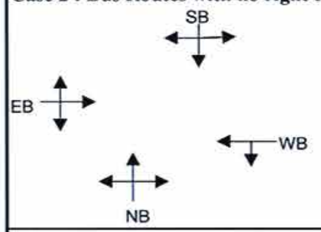
Figure 10.14.4

GUIDELINE FOR SITING OF BUS BAY ADJACENT TO CROSS-JUNCTION

A typical Cross-junction layout and the denotion indicated in the table below is shown in the Figure 10.14.

Assumptions made for this guideline :

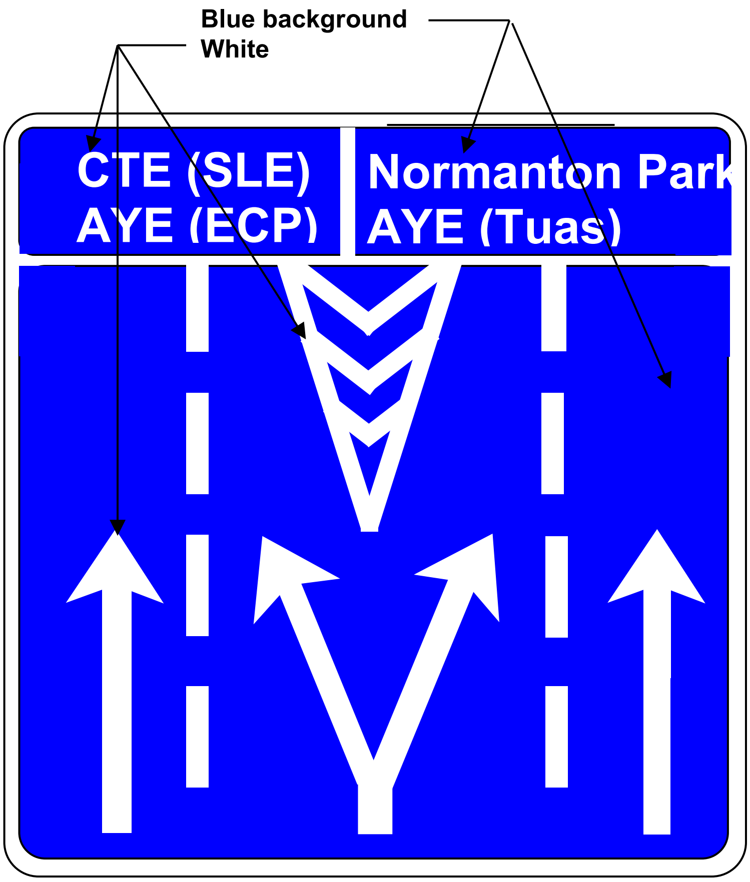
1. The road is a divided carriageway.
2. No site constraint.

Scenario	Minimum Dimension in metre				Remarks
	a [1]	b	c	d	
Case 2 : Bus Routes with no right turning at junction : 					
1) Bus bay provided :					
a) weave thru 2 lanes	30	0	$L_2 + d$	20	Yellow box across 1-lane of the carriageway to be provided. See Note 2.
b) weave thru 3 lanes	30	0	$L_2 + d$	20	Yellow box across 1-lane of the carriageway to be provided. See Note 2.
c) weave thru 4 lanes	30	0	$L_2 + d$	20	Yellow box across 1-lane of the carriageway to be provided. See Note 2.
2) Without bus bay :					
a) weave thru 2 lanes	50	0	$L_2 + d$	20	See Note 2.
b) weave thru 3 lanes	50	0	$L_2 + d$	20	See Note 2.
c) weave thru 4 lanes	50	0	$L_2 + d$	20	See Note 2.

Note:

[1] If the bus volume at the bus bay is high (exceed 120 buses/hr), double tier or higher capacity bus bay is required. LTA's Bus & Taxi Regulation Dept and Traffic Management will need to be consulted for dimension of a & d.

[2] LTA's Bus & Taxi Regulation Dept will need to be consulted and the Authority must agreed that there is no future bus route turning right in the Westbound direction.



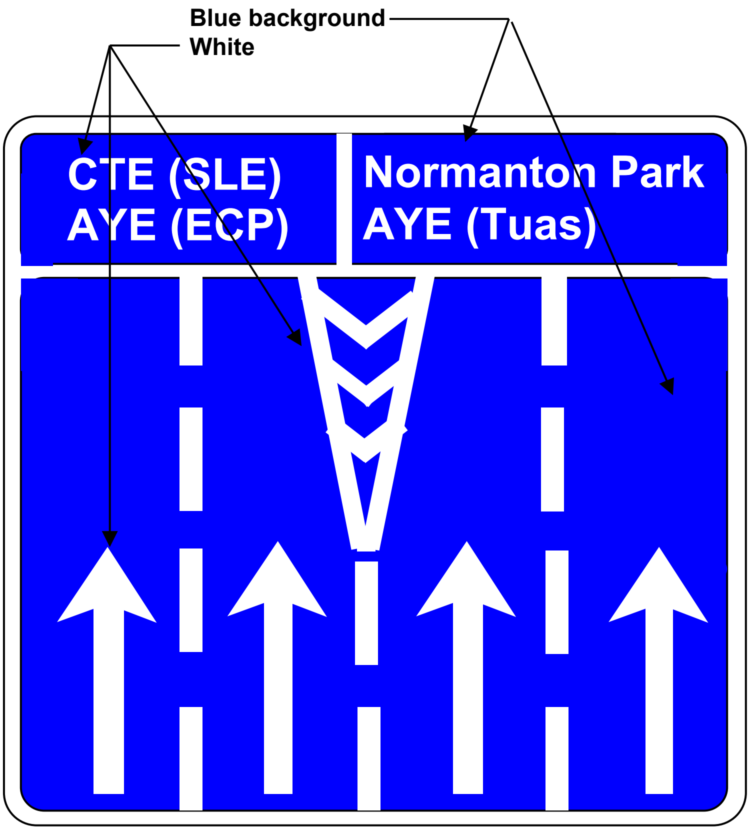


Figure 10.15

Lane Indication Signs

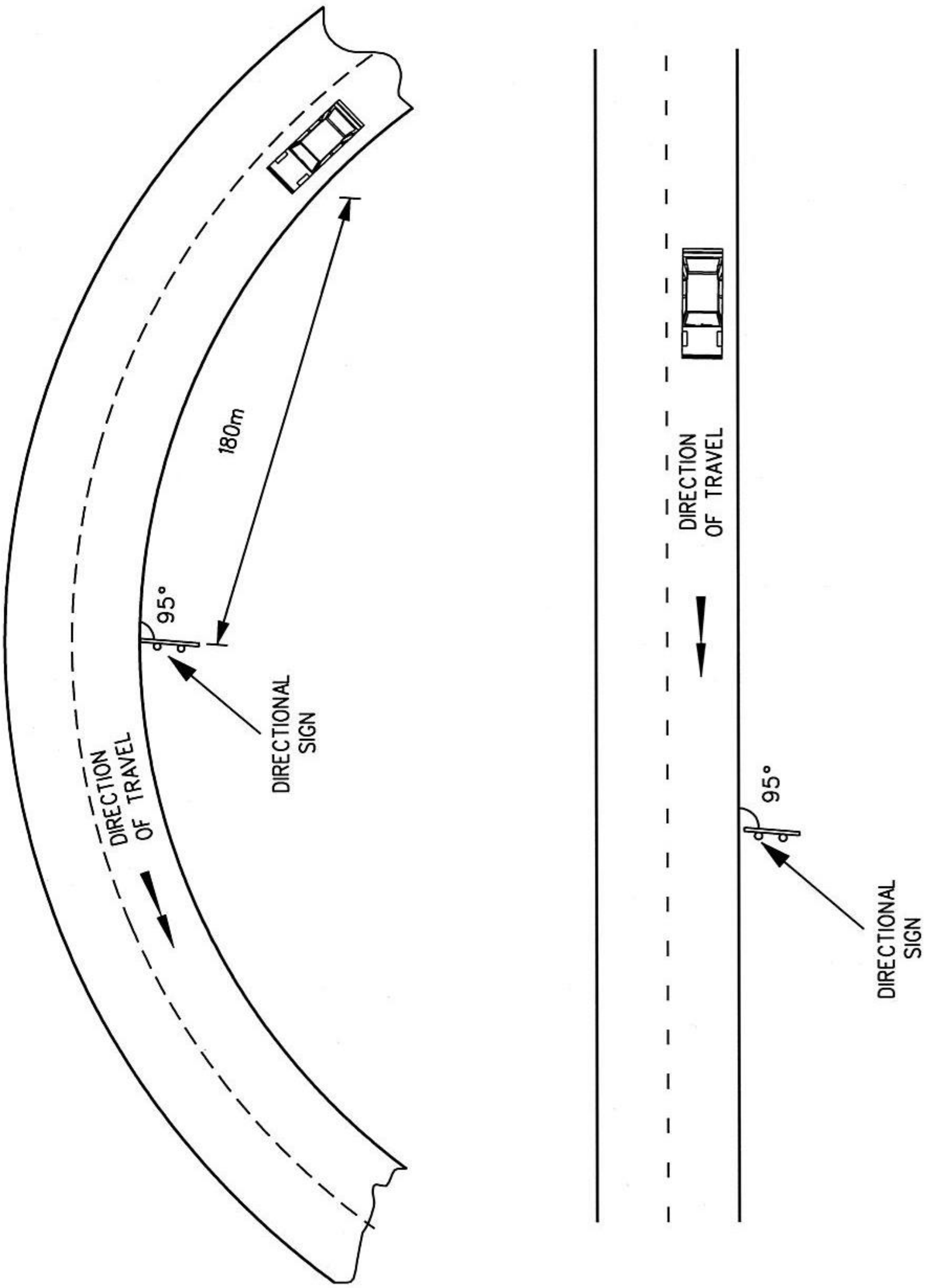
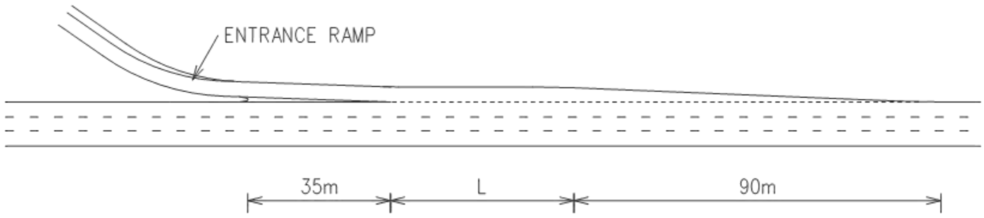


Figure 10.16

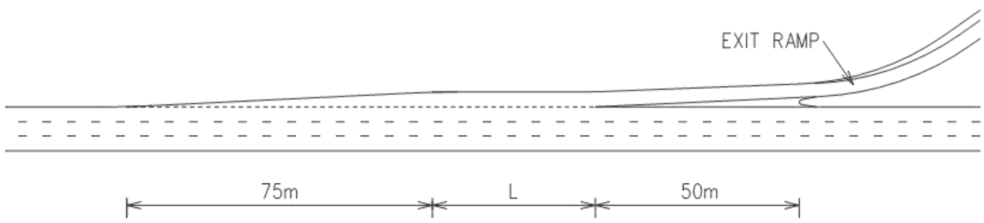
Typical Arrangement of Directional Sign

DESIGN SPEED OF ROAD ON THE MAIN CARRIAGEWAY (km/h)	MNIMUM LENGTH OF ACCELERATION LANE, L(m)					
	UPGRADE			DOWNGRADE		
	$G \leq 2\%$	$2\% < G \leq 4\%$	$4\% < G \leq 6\%$	$G \leq 3\%$	$3\% < G \leq 5\%$	$5\% < G \leq 6\%$
70	45	65	75	45	30	30
80	95	145	175	95	60	55



TYPICAL ACCELERATION LANE

DESIGN SPEED OF ROAD ON THE MAIN CARRIAGEWAY (km/h)	MNIMUM LENGTH OF DECELERATION LANE, L(m)					
	UPGRADE			DOWNGRADE		
	$G \leq 3\%$	$3\% < G \leq 5\%$	$5\% < G \leq 6\%$	$G \leq 2\%$	$2\% < G \leq 4\%$	$4\% < G \leq 6\%$
70	40	40	35	40	50	55
80	50	45	40	50	60	70



TYPICAL DECELERATION LANE

Figure 10.17

Acceleration and Deceleration Lane

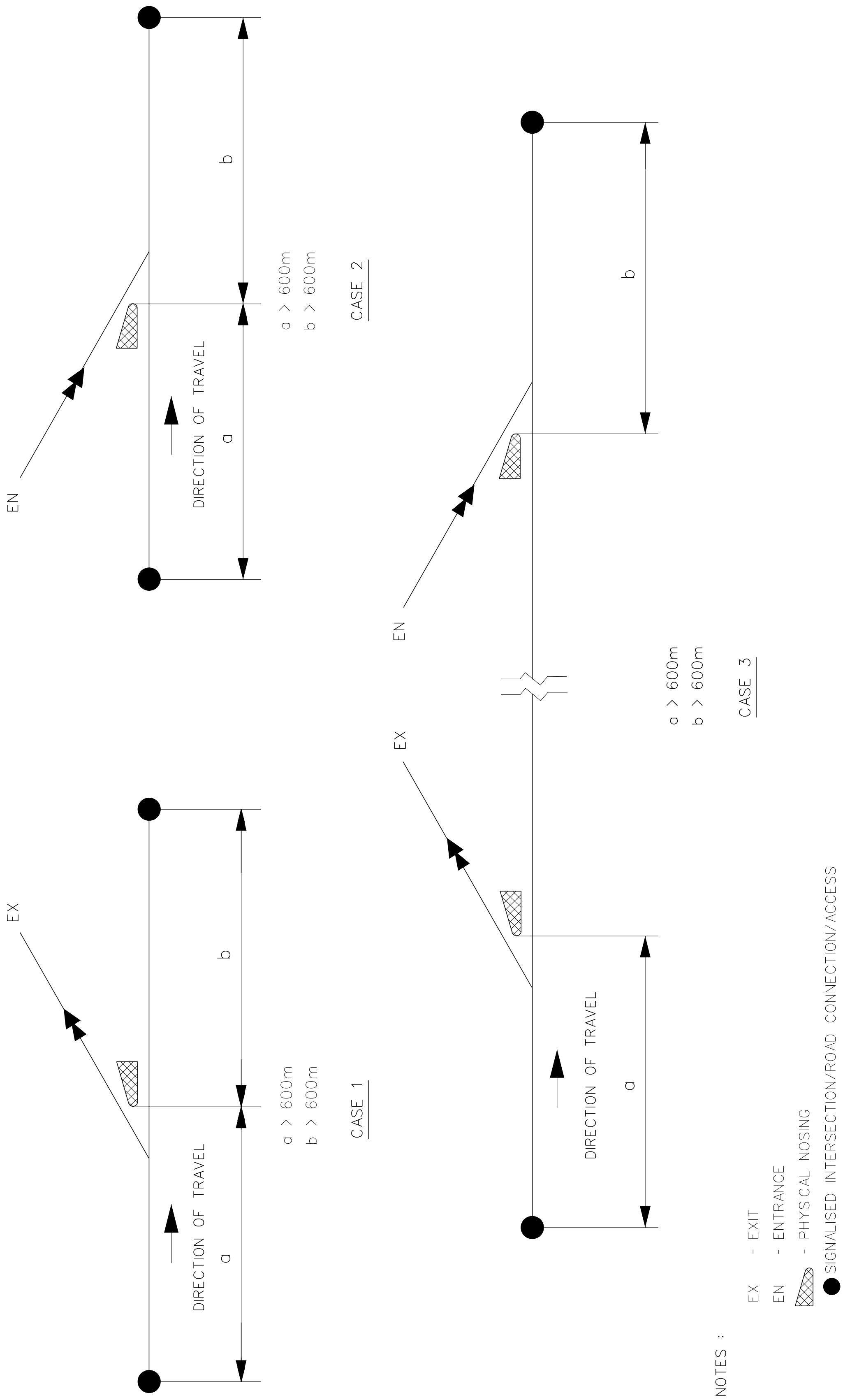


Figure 10.18

Conditions for the Provision of Acceleration and Deceleration Lane on Major Arterial Road

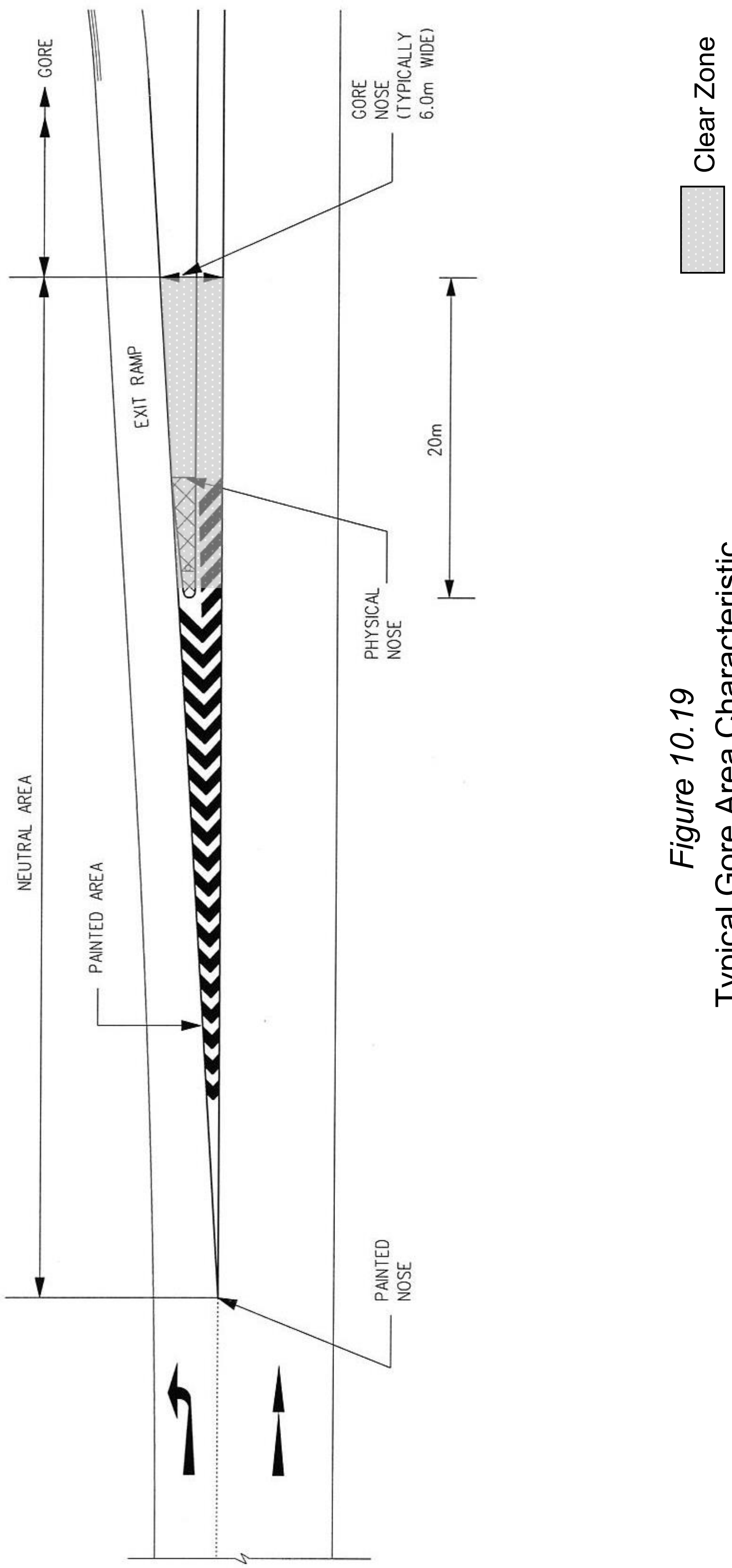
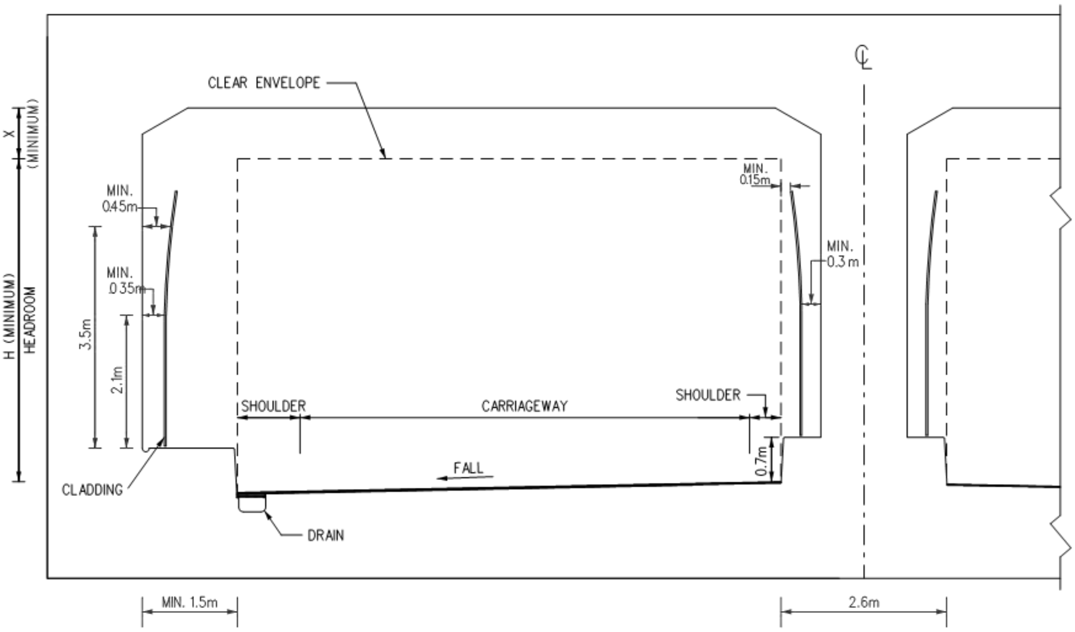


Figure 10.19
Typical Gore Area Characteristic

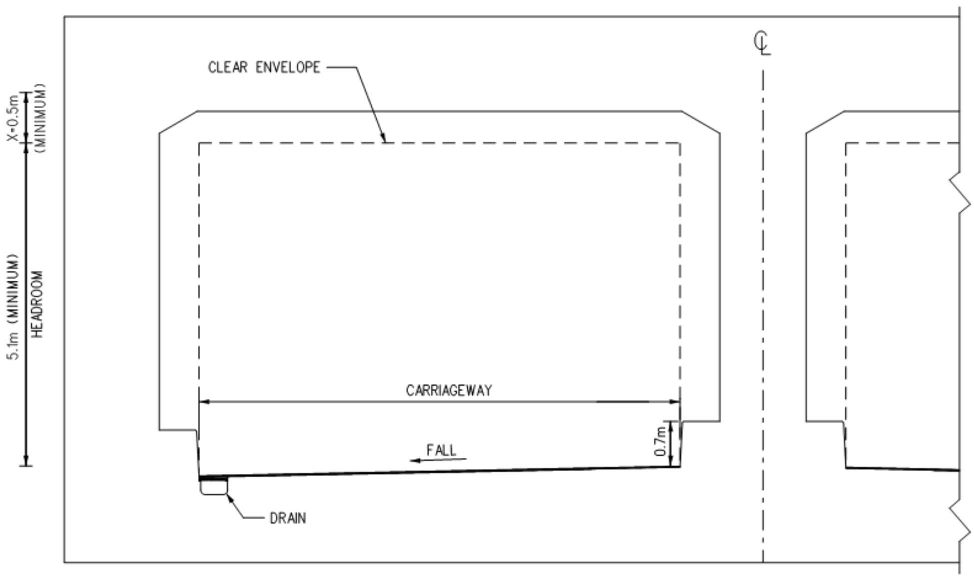


NOTE:

H = 5.1m AND X = 0.8m FOR TUNNEL (SIGNAGES, E&M SYSTEM).

H = 5.1m AND X = 0.5m FOR UNDERPASS (LIGHTING).

TUNNEL / UNDERPASS CROSS-SECTION FOR EXPRESSWAY
AND TUNNEL CROSS-SECTION FOR OTHER ROAD



NOTE:

X = SPACE FOR LIGHTING.

UNDERPASS CROSS-SECTION FOR OTHER ROAD

Figure 10.20

Cross-Section for Tunnel and Underpass

CHAPTER 11

STATION AND TUNNEL SERVICES FOR RAIL PROJECTS

11.1 GENERAL REQUIREMENTS

11.1.1 Standards, Codes and Regulations

The design of Station and Tunnel Services for Rail Projects shall be governed by all latest applicable local codes, regulations, standards and requirements issued by all the applicable local authorities and statutory boards including: -

(1) Public Utilities Board (PUB)
(2) Fire Safety and Shelter Department (FSSD)
(3) Productivity and Standards Board (PSB)
(4) Land Transport Authority (LTA)
(5) Building Control Authority (BCA)

11.1.2 Routing of Pipework and Services

Pipework shall comply with the requirements for water and electrical equipment in CDC Chapter 13, E&M Interface. Pipework, services and fittings shall be routed so as to accommodate future maintenance. As far as practicable, they shall not be located above escalators, tracks or voids.

11.2 DRAINAGE

11.2.1 General

11.2.1.1 All internal surfaces of structures shall be positively drained via channels, drains etc., either by gravity to existing storm water drainage or to wet sumps from where water shall be pumped to the storm water drainage to the approval of the appropriate local authorities and statutory boards and to the acceptance of the Engineer.

11.2.1.2 Separate drainage systems shall be provided for each of the following:-

(1) Tunnel Drainage

This system shall deal with water originating from:

- ground water seepage
- tunnel washing
- testing and emptying of the fire mains

- condensate from train air-conditioning
- rain water blown into the tunnel or brought into tunnel by rolling stock.

(2) Station Drainage

This system shall deal with water originating from:

- ground water seepage
- tunnel washing
- testing and emptying of the fire mains
- condensate from train air-conditioning
- condensate from ECS associated plants
- water tank overflow and drainage
- all clean water in the station
- all clean water brought into station

(3) Storm Water Drainage

This system shall deal with all rainwater falling on the station roofs and external areas to be directed into the existing storm water drainage system.

11.2.2 Tunnel Drainage

11.2.2.1 Seepage

Drainage design shall be based on permitted seepage values given in CDC Chapter 7, Bored Tunnels & Related Works. All water shall be directed so that the rails and rail fixings remain dry.

11.2.2.2 Tunnel Washing

The maximum discharge rate of 10,800 litres/hr of the tunnel washing wagon shall be allowed for in the design of the tunnel sump and pump.

11.2.2.3 Fire Main

The volume of water discharged during testing and emptying of the fire main shall be determined in co-ordination with the System-wide Contractor and allowed for in the design of the tunnel pump sump.

11.2.2.4 Condensate from Train Air-Conditioning

Condensate from moving trains may be assumed to be evaporated.

11.2.2.5 Tunnel Pump Sump

11.2.2.5.1 Location

A pump sump shall be located at every low point within each running tunnel. If the pump sump location coincides with a cross passage, then only one pump sump shall be provided and it shall be located in the cross passage.

11.2.2.5.2 Details

Provisions and layout of the pump sump shall follow that for the station as described in Clause 11.2.4.

The discharge pipes from the tunnel sump pumps shall be routed directly to the surface drains via the nearest station, vent shaft, escape shaft or service shaft/duct. The water shall not be discharged to another drainage pumping system within the Works. Swan necks shall be provided at the appropriate locations. Flap valves shall be provided at the discharge ends. Discharge pipe shall have a minimum diameter of 80mm. Velocity of water in discharge pipes shall be between 1m/s and 2.4m/s to ensure self cleansing and prevent scouring.

Tunnel drainage sumps shall be monitored at the nearest station.

11.2.2.5.3 Design and Construction

The design of the tunnel sumps and the pumps shall be in accordance with Clause 11.2.5.

The structural design of sumps shall comply to requirements in CDC Chapter 7, Bored Tunnels & Related Works and CDC Chapter 8, Underground Structures with the sumps regarded as an underground structure.

11.2.3 Station Drainage

General requirements are summarised in Table 11.1. All outlets shall be discharged to the drainage system unless stated otherwise in the specifications. The Drainage System shall incorporate these requirements and shall be designed in accordance with Clause 11.2.5. Design of Station Drainage systems shall comply with all requirements that may be imposed by the PUB.

TABLE 11.1 DRAINAGE SYSTEM PROVISION IN STATIONS

Room/Area	Requirement	Remarks
1. Seepage channel	Floor wastes at 10m max. interval	See Clause 11.2.3.1.
2. Escalator pit	Floor waste	See Clause 11.2.3.3
3. Vent Shaft/Duct	Floor waste	See Clause 11.2.3.4
4. Underplatform Areas	Channels, floor wastes	See Clause 11.2.3.7
5. All ECS associated Plant Rooms	Floor wastes	Adjacent to AHU for condensation water. See Clause 11.2.3.8
6. Lift pit	Sump	See Clause 11.2.3.9
7. Entrance stair	Floor waste per entrance	See Clause 11.2.3.10
8. Cable Trench, Valve Chamber & Maintenance Pits*	Floor waste	See Clause 11.2.3.11
9. All water tank/pump room	One sump with floor waste	For draining of tank water and overflow. See Clause 11.2.3.5
10. Adjacent to hosereel	One floor waste	For accidental spillage of water or fire fighting water. Drainage via seepage channel outlets is not allowed.
11. All enclosed staircases (at lowest landing)	One floor waste	For accidental spillage of water or fire fighting water.
12. Elevated and exposed platforms (elevated station)	Two parallel channels run along the track direction for each platform, with appropriate connection to the drainage system.	For rain water blown onto the platform plus washing water.
13. Planter in station	(a) Floor waste	Hydroponic planting discharge via a silt trap.
	(b) Surface drain	For ordinary planter.

* These are civil defence facilities in CD stations.

11.2.3.1 Seepage

11.2.3.1.1 Seepage drainage channels shall be provided at the floor level along the internal sides of all earth-backed external walls. A drainage channel of 100 mm diameter shall be formed and laid to fall to not less than 1 in 200. Discharge outlets (floor waste) of not less than 100 mm diameter shall be situated at not more than 10m centres.

11.2.3.1.2 Seepage drainage channels shall be lined with a suitable waterproofing membrane. Drainage channels, weepholes and outlets shall not pass through fire rated compartment wall.

11.2.3.1.3 Seepage drainage channels in floor finishes shall be at least 35 mm deep. A cavity of limited height shall be constructed to contain the drainage at the bottom of the wall if sufficient depth is not available in any floor finishes.

11.2.3.2 Cavity Walls

11.2.3.2.1 For the purpose of establishing cavity wall requirements are categorised as described below, and as listed in Table 11.2. For any rooms not listed therein, proposals shall be submitted and agreed with the Engineer. Where required in any given rooms/areas, cavity walls need to be provided only along the external walls.

(1) Category I

Generally, these are rooms/areas containing sensitive E&M equipment requiring extra protection from damp and moisture. Full height cavity walls shall be provided.

In addition, in rooms/areas that are accessible to the public, full height cavity walls shall also be provided to protect the architectural finishes from seepage water.

(2) Category II

These are rooms/areas without an immediate need for cavity walls but which requires provision for installation of such walls in the future. Ample space shall be provided in these rooms/areas for future cavity wall construction and the sizing of the rooms shall take this into account. All services/equipment mounted onto the earth backed wall in such rooms/areas shall be designed such that it can be easily removed and mounted onto a cavity wall should the need arise. Ceiling services shall also be such that it will not obstruct the future construction of the cavity wall.

(3) Category III

Cavity wall is not required.

11.2.3.2.2 For full height cavity walls, it shall be constructed as an inner lining with a seepage drainage channel confined to inside the cavity. Access panels of minimum 600mm x 600mm size placed at intervals not exceeding 10 metres shall be provided on the cavity walls to permit inspection and maintenance of the drainage system. The actual position shall be coordinated and located near a floor waste.

TABLE 11.2 GUIDELINE FOR CAVITY WALL ROOM CATEGORISATION

CATEGORY I	
1. Local Sequential Control (LSC) Room	
2. Main Distribution Frame (MDF) Room	
3. Communication Equipment / Integrated Supervisory Control System (CE/ISCS) Room	
4. Coaxial Distribution Room (CDR)	
5. Signal Equipment (SE) Room	
6. Passenger Services Centre (PSC) / Maintenance Sub-Centre	
7. Telecommunications Equipment Room (TER)	
8. Store, Office and staff toilet	
9. All Public Areas (including toilets)	
10. RC Drinking Water Tanks – Subject to conditions specified in Clause 11.2.3.6	
CATEGORY II	
1. Lift Motor Room (LMR)	2. Fare Equipment Room (FER)
3. AHU Room	4. Emergency Switch Room
5. Underplatform Exhaust Fan (UPE) Room	6. Uninterruptible Power Supply (UPS) Room / Emergency Power Supply (EPS) Room
7. Permanent Way Store	8. Service Transformer (service TX) Room
9. Fire Pump Room	10. Fuel Pump Room
11. DB Room / AFC-DB	12. Traction Transformer (Traction TX) Room
13. Generator Room	14. Traction Power Sub-Station (TPSS)
15. Fuel tank	16. Clean Gas Room

TABLE 11.2 (CONT'D)

17. 22kV Switch Room / 22kV Switch Room (intake)	18. LV Switch Room
19. CD Cooling Tower Room	20. Tie Breaker Room
21. Water distribution Control room	22. CD First Aid Room
23. ECS Plant Room	24. Fireman's and Escape Stair
25. Fan Room	26. CD General Store
27. Permanent Way Store	28. Air Intake Plenum
29. PUB Intake Monitoring Kiosk	30. Intake Air Lock
31. Smoke Extract Fan (SEF) Room	32. CD Equipment Store
33. Tunnel Ventilation Fan (TVF) Rooms	34. CD Pantry
35. ECS Control Room	
CATEGORY III	
1. Ventilation Shafts	2. CD sliding door chamber
3. Lift Shafts	4. Trackside wall
5. Ventilation Passageways	

11.2.3.3 Escalator Pits

11.2.3.3.1 The escalators serving the entrances shall have the lower escalator landing pits graded to a minimum fall of 1:200 towards the floor trap/waste connecting to the sewerage/drainage system. The rest of the escalators shall have a dry sump at the down stream of the lower escalator landing adjacent to the escalator pit which shall be accessible outside the escalator truss.

11.2.3.3.2 For stations not located within a water catchment area, water in the escalator pits shall be discharged to the drainage system. If the station is located within a water catchment area, then it shall be discharged into the sewerage/sanitary plumbing system.

11.2.3.4 Vent Shafts/Ducts

Vent Shafts shall be detailed such that rainwater is neither drained nor blown into them. Provision shall be made at the ventilation shaft terminals to drain all rainwater into the drainage system. The base of the vent shafts/ducts shall not be allowed to accumulate any water and shall be graded to fall towards the floor waste at a gradient of 1:200 minimum.

11.2.3.5 Water Storage Tanks

11.2.3.5.1 The storage capacity of a water tank is the volume of water that can be drawn by the water booster/transfer pumps. Water tanks shall be designed to minimise dead water volume.

11.2.3.5.2 The inlet and outlet pipes of the tank shall be located to avoid stagnation of water in the tank.

11.2.3.5.3 The base of the water tank shall have a minimum fall of 1:200 towards a 200mm minimum diameter drain-off pipe with puddle flange. This shall be installed at the base of the water storage tank to drain the water storage tank completely.

11.2.3.5.4 Overflow and drain-off water from storage water tanks shall not be discharged into the seepage channels directly. They shall be discharged directly to either a drainage sump or a pump sump.

11.2.3.6 R.C. Drinking Water Tanks

RC water tanks for drinking water shall not be in direct contact with soil. Where this is not possible, a cavity of minimum 100 mm with seepage channels shall be provided between the water tank and the earth-backed wall/slab to prevent contamination from any source. Floor waste for the seepage channel shall not exceed 10m intervals. Sufficient provisions for future maintenance of the seepage channel and floor waste shall also be provided.

11.2.3.7 Underplatform Areas

Drainage channels of minimum width 100mm shall be provided in the underplatform areas with a minimum gradient of 1:200. Discharge outlets (floor waste) of not less than 100 mm diameter shall be situated at not more than 10m centres. The floor finishes of the underplatform areas shall be made to fall towards these channels.

11.2.3.8 Condensate Drainage

11.2.3.8.1 Drainage is required for condensate water from all ECS associated systems (e.g., AHU Plant Room, ECS Plant Room etc.) including air conditioning from trains, ancillary and commercial areas. All condensate water shall discharge to the drainage system unless stated otherwise in Clause 11.3.

11.2.3.8.2 The location of the condensate points and volume of discharge shall be determined through co-ordination with the system-wide contractor. Floor trap being provided shall not be used for discharging water used for the cleaning of ECS equipment.

11.2.3.8.3 Any exposed condensate drain/pipe (including floor trap) beneath the suspended floor slab shall be insulated to prevent condensation.

11.2.3.9 Lift Pit

The base of lift pit shall be graded to have a minimum fall of 1:200 towards the sump of 450mm by 450mm by 450mm as per the requirements under CP 2: The Installation, Operation and Maintenance of Electric Passenger and Goods Lifts.

11.2.3.10 Entrances

A cut off drain shall be provided across each entrance, at the top of the stairway and escalator. The collected water shall be discharged to the nearest surface drain. The structural recess for the cut off drain shall be 240mm wide with a minimum depth of 200mm deep, covered with a stainless steel grating. A catch pit shall be provided at the drain outlet, or as close thereto as can be arranged, to prevent debris from entering the drainage run. Where pipes pass through the joints between the station structure and the entrance, they shall be detailed to accommodate all movements.

The minimum platform level and crest protection level for all entrances, exits, linkages etc. shall be in accordance with PUB's latest requirements.

11.2.3.11 Cable Trenches, Valve Chambers and Maintenance Pits

The top of the cable trenches, valve chambers and maintenance pits shall be made to fall away to prevent water from flowing inside. The floors of cable trenches, valve chambers and maintenance pits shall be graded to have a minimum gradient of 1:200 towards the floor waste.

11.2.3.12 Movement Joints

Structural movement joints are highly susceptible to water leakage and adequate provisions for the collection and discharge of all water leakage shall be provided.

11.2.3.13 Dry Sump

A dry sump is a drainage sump without any outlet. Dry sumps shall not be provided unless accepted by the Engineer.

11.2.3.14 Drainage Sump

Where the direction of a horizontal pipe changes or where two horizontal pipes merge, a drainage sump shall be provided unless the pipes are accessible from below. Pipes that are buried or cast in concrete, or located above the track are considered inaccessible from below. The design and provision of drainage sumps shall comply with sewerage and sanitary plumbing requirements.

Water in a drainage sump shall not be channelled such that its direction of flow turns through an angle of more than 90^{o}.

11.2.4 Station Pump Sumps

11.2.4.1 Location

11.2.4.1.1 Station pump sumps shall be located to suit the individual requirements of each station and to be located at strategic locations such that the total number of pump sumps is minimised. The utilisation of every drainage pump sump shall be maximised to collect water from as many sources as possible.

11.2.4.1.2 Sumps shall be accessible for inspection and maintenance at all times. Pump sumps shall as far as possible not be located at the passageway or corridor. Each pump sump shall be located in a dedicated sump room.

11.2.4.2 Details

11.2.4.2.1 Provisions shall be made in the design of the sumps for the discharge mains, power supply and monitoring cables to the pumps.

11.2.4.2.2 Check valve of single flap type and a gate valve shall be provided on the discharge main of every pump. They shall be located above the sump such that they are accessible without the need to enter the sump and after removal of the access cover.

11.2.4.2.3 The layout shall be such as to facilitate easy removal and replacement of pumps without entering the sump. Lifting facilities (e.g., overhead runway beam, eye bolt etc.) and equipment shall be provided to enable easy lifting of the pumps. Adequate removable chain blocks shall be provided.

11.2.4.2.4 Access openings shall be provided directly above the pumps for easy installation and removal of the sump pumps. Access openings shall be fitted with aluminium chequered covers and provided with aluminium access ladders with extensible handhold up to 1150mm above access cover level. Ladder requirements are described in Clause 11.5.

11.2.4.2.5 For each sump pump, a control panel shall be placed at a convenient, easily accessible location and shall be constructed with a waterproof type enclosure.

11.2.4.2.6 A stainless steel screen shall be provided in the pump sump of every drainage sump immediately upstream of the pump sump. The location of the screen shall be such that a maintenance worker standing at the access cover level can easily clear all debris trapped.

11.2.4.2.7 The discharge pipes from the sump pumps shall be routed directly to the surface drains via the nearest and shortest route (e.g., vent shaft, service shaft/duct, entrance etc.). The water shall not be discharged to another drainage pumping system within the Works. Swan necks shall be provided at the appropriate locations. Flap valves shall be provided at the discharge ends. Discharge pipe shall have a minimum diameter of 80mm.

11.2.4.2.8 The pump sumps shall be waterproofed using an accepted waterproofing admixture or alternatively, using an accepted liquid membrane applied on the interior surfaces of the sump walls.

11.2.5 Sump and Pump Design Directives

11.2.5.1 Pump Design

11.2.5.1.1 For each pump sump, a minimum of two pumps with similar capacities shall be provided, one of which shall be the standby pump. The duty pump(s) shall be capable of handling the full discharge requirements. The stand-by pump shall be automatically activated when the duty pump fails.

11.2.5.1.2 The design of the system shall be such that the number of starts/stops for each pump shall be limited to not more than 10 per hour. However, the motor starter shall be sized to 15 starts/stops per hour.

11.2.5.1.3 Controls shall be provided such that there is an automatic change over of duty and standby pumps during each cycle of operation. This is to enable even distribution of wear and tear of the pumps.

11.2.5.2 Design Pump Capacity

11.2.5.2.1 The design pump capacity, Q_p for each sump shall be calculated using the following:

Q_p = [(seepage + condensate)] x 2 + max[tunnel washing, fire main, tank overflow]

However, the design pump capacity shall also ensure that the velocity, *V* of water in the discharge pipes is between 1.0*m/s* and 2.4*m/s* (except for the *common* water tank discharge pipe where *V* may exceed 2.4*m/s* when both pumps are operating).

11.2.5.2.2 Water seepage shall be obtained based on the contribution area of tunnel lining or earth backed external walls using the seepage rates given in M&W Chap 16 and CDC Chap 8.

11.2.5.3 Pump Selection

In order to select the proper pump, the following parameters shall be considered during the selection of the pumps: -

(1) Design Pump Capacity

(2) Operating Head

(3) Efficiency

(4) Power Rating

(5) Discharge Pipe Diameter (Max. & Min. water velocity)

Having due regard to these criteria, pumps from at least two different manufacturers available in the market shall be identified and submitted together with the calculations. The sump shall then be sized to accommodate the largest pumps.

11.2.5.4 Sump Sizing

11.2.5.4.1 The size of each sump is determined based on the following parameters:

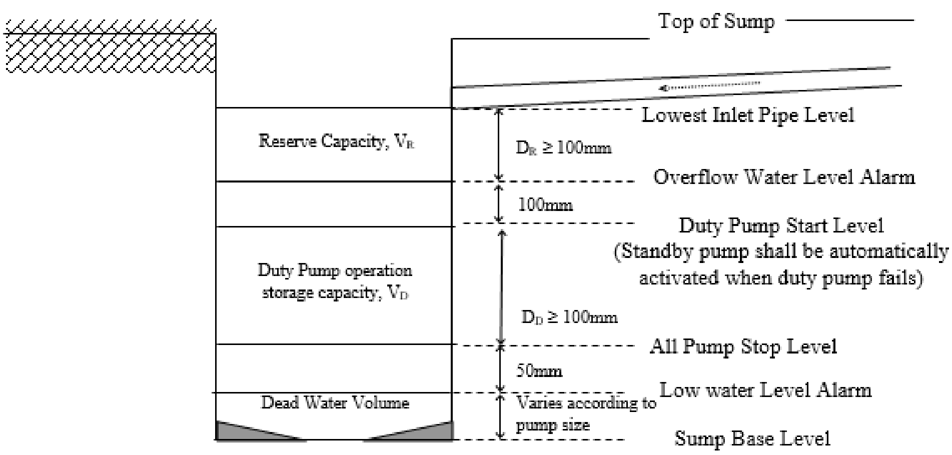
(1) Dead water volume;

(2) Duty pump operation capacity;

(3) Reserve capacity;

11.2.5.4.2 The typical operating water levels of a pump sump shall be as shown in Figure 11.1 below.

Fig. 11.1 Pump Operating Levels for Sump Sizing



11.2.5.5 Dead Water Volume

Dead water volume is measured from the Sump Base Level to the Low Water Level Alarm and shall be minimised by haunching the base of the sump.

11.2.5.6 Duty Pump Operation Storage Capacity

The Duty Pump Operation Storage Capacity is measured from the All Pump Stop Level to the Duty Pump Start Level. The volume shall be computed from:

$$V_D = \frac{Q_{select}\, x T}{4}$$

where Q_{select} is the higher flow capacity of the selected pumps (see above).

T is the time between 2 sequential starts (i.e., one complete start-stop cycle) and is computed from:

$$T = \frac{3600}{(No.\ of\ Start/Stops\ per\ Hour)} \text{ seconds}$$

Total No. of Starts/Stops per Hour shall not exceed 10 and the level between the All Pump Stop Level to the Duty Pump Start Level shall not be less than 100mm.

11.2.5.7 Duty Pump Start Level to Overflow Water Level Alarm

The level between the Duty Pump Start Level and Overflow Water Level Alarm shall be set at 100mm.

11.2.5.8 Reserve Capacity

11.2.5.8.1 The Reserve Capacity shall be measured from the Alarm Level to the Lowest Inlet Pipe Level. It shall be computed from:-

$$V_R = [(Seepage\ x2) + Condensate\ (from\ stationary\ trains)]\ x$$
$$(response\ time\ of\ 6hrs\ or\ 24hrs\ depending\ on\ accessibility)$$

The level between the Overflow Water Level Alarm and Lowest Inlet Pipe Level shall not be less than 100mm.

11.2.5.8.2 For Tunnel pump sumps, the response time shall be 24 hours.

For Station pump sumps, the response time will be taken as 24 hours if the sump is not accessible during train operation, and 6 hours otherwise.

11.2.5.8.3 Condensate water from stationary trains and station air conditioning plant, ancillary and commercial areas shall be determined through co-ordination with the system-wide contractor.

11.2.5.8.4 Accident/emergency inflow such as water tank overflow and occasionally large but manageable inflows such as track washing and fire main draining need not be considered in the computation of reserve capacity. Condensate from moving trains may be assumed to be evaporated.

11.2.6 Storm Water Drainage

Design of storm water drainage run off shall be in accordance with the Code of Practice for Surface Water Drainage, and the latest Surface Water Drainage Regulations issued by the PUB (Drainage) and any other requirements that may be imposed by PUB.

11.2.6.1 Run Off

The Rational Formula shall be used for determining surface water run off. The coefficient of run off for cutting slopes and track area shall be taken as 1.0.

Run-off from neighbouring lot or adjacent land if affected shall be diverted by new drains constructed to the approval of PUB(Drainage).

11.2.6.2 Roof Drainage

11.2.6.2.1 Roof drainage shall be designed and constructed to dissipate water from the roof by the most effective and direct route possible to the surface drains. Design of roof drainage shall be in accordance with all the latest PUB regulations and with the latest edition of SS CP26 Code of Practice for Drainage of Roofs. Rainwater outside the station shall not be drained into the Station/Tunnel pumped drainage system.

11.2.6.2.2 Rainwater pipes and outlets provided to any flat roof shall ensure that the build up of water during a flash storm does not exceed 30 mm.

11.2.6.2.3 Rainwater pipes shall be of sufficient bore, and in long lengths with easy bends to ensure that a back up of water does not occur. The outlets to flat roofs shall be filled with dome grating or equivalent to prevent dirt, leaves or any foreign substance from blocking the down pipe.

11.2.6.2.4 Notwithstanding the above, flat roofs shall be provided with overflow facilities.

11.2.6.2.5 Wherever pitched roofs form part of the structure, adequate overhang of the roof shall be provided to prevent rainwater water falling onto people who are liable to stand or walk beside the building and the rainwater must be drained to the surface drainage system. Sufficient surface water drains shall be provided on the ground level to collect all water from the roof and prevent water ponding. Roof gutters shall be avoided as far as possible. Where it cannot be avoided, the gutters of sufficient capacity to collect rainwater from the roof without spillage shall be provided across the edge of the roof. Roof gutters shall be coated with an approved waterproofing membrane with minimum ten years warranty and roof outlets shall be properly dressed and sealed with a suitable membrane at the junction with the roof. Overflow pipes shall be provided at the ends of the gutters.

11.2.6.2.6 Rainwater downpipes shall be positioned at suitable distances to collect and discharge water to the stormwater drainage system. All gutters, outlets and downpipes shall be positioned outside buildings as far as possible so that if any leak occurs, no damage to sensitive equipment, or other inconvenience, can arise. Rainwater downpipes shall be fitted with screw-on rodding eyes at the base of the stack.

11.2.6.3 Paved Areas

All paved areas around the station shall be sloped to provide effective surface run-off. The slope shall be directed away from station entrances. Adequate cut off drains shall be provided and directed into the existing storm water drainage system.

Where perimeter drains and a surface structure are provided the drain shall be integral with the main structure to avoid differential settlement problems.

11.3 SEWERAGE & SANITARY PLUMBING

11.3.1 General

General requirements are summarised in Table 11.3. The sewerage and sanitary system shall incorporate these requirements and shall be designed in accordance with Clause 11.3.3. Design of sewerage and sanitary system shall also comply with all requirements that may be imposed by the PUB.

11.3.2 Design Code

The design of sewerage drainage system shall be in accordance with the latest PUB Sewerage Department Code of Practice on Sanitary Plumbing and Drainage System, Sanitary Plumbing and Drainage System Regulations, Sewerage Procedures and Requirements Handbook by PUB, the Code of Practice on Sanitary Facilities and Fittings for Public Toilets and the Code of practice on Pollution Control.

TABLE 11.3 SEWERAGE SYSTEM PROVISION IN STATIONS

Room/Area		Requirement	Remarks
1	Escalator pit	One floor trap	See Clause 11.2.3.3
2	All ECS associated Plant Rooms	Actual number of floor traps depend on equipment layout and room size.	Floor trap with 1 x 1 m kerbed floor basin to be located below water tap.
3	Cooling towers / exhaust plenum	One floor trap plus service channel if necessary	Floor trap with 1 x 1 m kerbed floor basin to be located below water tap, bleed off and overflow. For outdoor cooling tower area, a cover to be provided for the floor trap to prevent rain water ingress.
4	Sprinkler water tank/pump room	One floor trap	Floor trap with 1 x 1 m kerbed floor basin to be located below water tap. For washing water during cleaning of equipment
5	Water Distribution Control Room	One floor trap	Floor trap with 1 x 1 m kerbed floor basin to be located below water tap. For washing water during cleaning of equipment
6	Ancillary rooms, station control room	Floor traps where necessary	Preferred location for floor traps would be the corridor areas for condensate water.
7	Cleaner room	One floor trap	For washing water.
8	Refuse store	One floor trap	For washing water.

TABLE 11.3 (Cont'd)

Room/Area		Requirement	Remarks
9	Staff room	Floor trap	For wash basin.
10	Staff and public toilet	Floor waste/trap	For washing water.
11	Within WC cubical	Floor waste/trap	For washing water.
12	Commercial Areas (shops, kiosks etc.)	One floor trap per shop or kiosk.	Floor traps near taps.
13	*CD pantry	Floor trap	As per BCA guidelines.
14	*CD first aid room	Floor trap	As per BCA guidelines.
15	*CD decontamination room	Floor traps	As per BCA guidelines.
16	*CD Dry toilet	Sealed floor traps.	For waste from wash basin. The floor traps shall be covered with floor access plates during peace time. Location and spacing shall be as per BCA guidelines.

* These are civil defence facilities in CD stations. These facilities will only be used during CD operations.

11.3.3 Design Directives

11.3.3.1 Sewerage and Sanitary Plumbing System shall be constructed using approved materials, laid to fall in accordance with PUB's requirements.

11.3.3.2 Wherever possible sewage shall be gravity fed into the main sewerage network.

11.3.3.3 The pipe work where fixed to walls or under slabs shall be secured with brackets and hangers capable of supporting the fully loaded pipes.

11.3.3.4 On vertical stacks and on all branch connections rodding eyes shall be fitted to bends not exceeding 135° or at connections between pipes of different material.

11.3.3.5 All pipes shall be properly caulked and sealed at the joints and shall be to PUB's approval.

11.3.3.6 The PUB approved type of autosensing/electronic sensor flush valves shall be provided for water closets and urinals at public toilets and staff toilets. Where water pressure is inadequate for the operation of these flush valves, water tanks with booster pumps shall be provided. Sufficient and adequate power supply shall be available to operate these flush valves.

11.3.3.7 Penetrations through beams, slabs or any structural members for waste pipes shall be avoided as far as possible.

11.3.3.8 Floor traps/wastes shall be placed with consideration to water areas/points for which the provision is made. Floor traps shall not be obstructed or covered in any manner to hinder access to maintenance personnel.

11.3.3.9 The floor of all ECS associated plant rooms (e.g. ECS Plant Room, AHU Plant Room etc.) shall be waterproofed using an accepted liquid applied membrane.

11.3.4 Sewage Pump Sumps

The design of sewage pump sumps shall be similar to that of the drainage pump sumps except that they collect and dispose waste water.

11.3.5 Sewage Ejector

11.3.5.1 Sewage ejectors shall be provided at the base level of underground stations for the collection and disposal of sewage from staff toilets, crew toilets, public toilets and waste water from plant rooms/areas. All sewage and waste water from ground level or above shall be directly discharged into the main sewerage network by gravity as far as possible.

11.3.5.2 Sewage pump sumps shall be located to suit the individual requirements of each station and to be located at strategic locations such that the total number of pump sumps is minimised. The utilisation of every sewage pump sump shall be maximised to collect water from as many sources as possible.

11.3.5.3 For CD Stations, no ejector/sewage pumping system shall be designed to serve only CD facilities. Facilities that operate only during CD Mode (e.g., dry toilet) will result in low usage of the ejector/sewage pumps and increase maintenance burden in the future. Hence, any ejector/sewage pumping system shall be designed for either peace time facilities or for both CD and peacetime facilities.

11.3.5.4 Ejector pumping system shall be located in rooms specifically designated for this purpose and with direct access facilities for maintenance. A minimum of two pumps (duty & standby pump) and an ejector tank shall be provided for each ejector system. The rooms shall be minimum 3m x 3m to comply with PUB’s requirement.

11.3.5.5 Access openings shall be provided directly above the ejector tanks and pumps for easy removal/replacement. Access openings shall be fitted with aluminium chequered covers and provided with aluminium alloy access ladders with extensible handhold up to 1150mm above access cover level. Ladder requirements are described in Clause 11.5.

11.3.5.6 Lifting facilities (e.g., overhead runway beam, eye bolt etc.) and equipment shall be provided to enable easy lifting of the ejector tanks/pumps. Adequate removable chain blocks shall be provided. Sufficient clearance all around the sewage ejector shall be provided for ease of maintenance.

11.3.5.7 Containment of over spillage of sewage from ejector tank shall be considered. A sewage sump pump shall be provided in every ejector room and connected to the ejector pumping main.

11.3.5.8 For both ejector and sewage pumps, the minimum and maximum flow velocities allowed in the discharge pipe shall be 1.0m/s and 2.4m/s respectively. This is to ensure self cleansing velocity and to prevent scouring of pipes. For each ejector/sewage pumping system, the pumps shall be designed for the peak sewage/waste water inflow generated.

11.3.5.9 Discharge pipework of minimum 100mm diameter shall consist of check valve of single flap type and a gate valve. They shall be located above the ejector tanks such that they are accessible without the need to enter the ejector sump and after removal of the access cover.

11.4 WATER SERVICES

11.4.1 General

11.4.1.1 The design of water supply to stations shall comply with the latest Singapore Standard CP 48 for Water Services and Public Utilities (water supply) Regulations.

11.4.1.2 Underground piping shall be laid at such a depth that it is unlikely to be damaged by traffic loads and vibration. Where piping has to be laid in any ground liable to subsidence then special consideration shall be given to the type of piping to be used and the type of joint to be adopted in order to minimise risk of damage due to settlement. Where piping has to be laid across recently disturbed ground, continuous longitudinal support shall be provided.

11.4.1.3 Provision shall be made at every bend, branch and dead end in a main to resist the hydraulic thrust.

11.4.1.4 The PUB’s water meters chamber shall be provided above ground level. Wherever possible, a separate chamber to house the double check valve assembly for the fire fighting line shall be provided and located below ground level if it is technically feasible with a drainage facility to prevent flooding of the underground chamber. Location of the water meter chamber shall be easily accessible for meter reading and unobstructed, and shall be approximately 1m from the site boundary.

11.4.1.5 Common water distribution pipes shall not be routed in commercial/retail areas.

11.4.1.6 All valves and taps must be accessible for service and maintenance. Where valves or taps are installed above the ceiling or behind walls (seepage walls etc.) appropriate access panels must be provided in the ceilings or walls for maintenance and repair of the valves and taps. Control valves shall be provided at strategic and easy access location. Adequate brackets supports must be provided to the supply pipe in the vicinity of the bends.

11.4.1.7 Water supply to the stations falls into the following categories:

(a) a potable water system

(b) a water system for fire fighting

(c) a Civil Defence (CD) water supply where applicable

(d) a cooling water make-up system for ECS

(e) a water supply system for plant and equipment operation

(f) a water supply system for trains/plant/machinery/ equipment washing

(g) a water system for irrigation

11.4.2 Water Supply System

11.4.2.1 Drinking water tank/pump room shall not be located next to an ejector room or toilet or any potentially polluted area.

The areas/rooms in stations that require water supply provisions are given in Table 11.4.

TABLE 11.4 WATER SUPPLY PROVISION IN STATION

Room/Area	Requirements	Remarks
1. Sprinkler Water tank/Pump room	One tap	
2. Cleaner Room	One tap with sink	
3. Refuse Store	One tap	

Room/Area	Requirements	Remarks
4. Bin Point/Bin Centre	One tap	
5. Staff Room	One tap with sink	
6. Toilets	Taps as necessary	Taps in WC cubicles with squatting pans to meet PUB requirements.
7. Planter in Stations	Taps as necessary	Liaise with PUB/Nparks to meet their specific requirements, if any.
8. *CD Pantry	As per BCA guidelines.	
9. *CD First Aid Room	As per BCA guidelines.	
10. *CD Decontamination Room	As per BCA guidelines.	
11. *CD Dry toilet	As per BCA guidelines.	
12. All ECS associated Plant Rooms	Taps as necessary	- Near condensing unit. - Near AHU. - Near filter.
13. Ejector Room	One tap	
14. Drainage Pump Room	One tap	
15. Water Distribution Control Room	One tap	
16. Cooling Tower Room/Area	One tap	
17. Commercial Areas (shops, kiosks etc.)	One tap off point	

* These are civil defence facilities in CD stations. These facilities will only be used during CD operations.

11.4.2.2 Water Storage Capacity shall be provided at all Stations to ensure that, the air conditioning system can remain operational at normal load for one operational day (approximately 18 hours) in the event of a loss of direct water supply.

11.4.2.3 Water Storage Capacity shall be provided at all Depots to ensure that, in the event of a loss of water supply for up to 24 hrs;

- The air conditioning systems to all rooms containing equipment essential to the operation of the railway system and to the operational control room can remain operational at normal load;
- Water related facilities (e.g., toilets) for the key operational personnel and their managers in the Control Centre and relevant Offices can remain operational;
- Any other facilities essential to the operation of the depot can remain operational.

The water supply to these facilities shall be a separate supply through a dedicated storage tank, so that “ramping down” of other non essential facilities will not be taken from this supply.

11.4.2.4 Water booster system shall be provided with secondary power supply.

11.4.3 Water System for Fire Fighting

If a fire engine access road is provided, every part of the fire engine access road shall be within an unobstructed distance of 50m from a hydrant. Where a public hydrant is not available, private hydrant(s) fed after the water meter shall be provided in accordance with SS CP29 and PUB Regulations.

In all cases, a 100mm x 150mm branch line tee-off after the water meter shall be laid to supply water directly to the sprinkler water tank.

Fire hydrants shall be supplied and installed by the Contractor.

11.5 ACCESS LADDERS

11.5.1 General

Access ladders shall be provided in the following instances:-

(a) Access to equipment where regular maintenance works is required.

(b) Escape or emergency exit purposes.

(c) Any other situations where it is essential to provide one. (e.g., underplatform voids in stations, air shafts etc.)

11.5.2 Design

The design shall conform to Parts 1 to 3 of BS EN ISO 14122. All fixings to the structure shall be designed in accordance with CDC Chapter 8, Underground Structures. The minimum bolt diameter shall be M12.

Extendable type handrails shall be provided where it is not practical to extend side strings above the landing level to form handrails (for example sump pits in cross passages where pedestrian access is required over the sump pit). They shall be provided at each string and shall be capable of being temporarily extended to provide temporary handrails.

11.5.3 Material

The material shall be stainless steel or aluminium.

CHAPTER 12

EXTERNAL WORKS

12.1 PAVED AREAS

12.1.1 Paved areas around structures shall be designed to withstand the loads likely to be experienced during delivery and removal of equipment. Where fire engine access is required, the area shall comply with the requirements given in Singapore Civil Defence Force (SCDF)'s Code of Practice for Fire Precautions in Rapid Transit Systems.

12.1.2 Movement joints shall be provided to prevent cracking of finishes. These joints shall be provided in accordance with the Architectural Materials and Workmanship Specification.

Movement joints shall be provided at the interface of non-suspended and suspended areas. Differential settlements at these joints shall not be a hazard to persons using the areas and in no case shall exceed 6 mm.

12.1.3 Maximum long-term allowable settlement in the pavement structure around the station shall not exceed 20 mm. The differential settlement between any two adjacent points within a panel shall not exceed 1 : 1000 subject to a maximum of 15 mm. The differential settlement between any two adjacent panels shall not exceed 6 mm. All external paving shall be designed to drain off surface water efficiently to prevent ponding of water.

12.2 IRRIGATION SYSTEMS AND LANDSCAPING

12.2.1 Landscaping and irrigation system, if required shall comply with the requirements of the National Parks Board (NParks), and all other relevant government agencies. Each irrigation system shall be proven to be workable by means of a trial of adequate time period.

12.2.2 Landscaping or turfing shall be provided on both non-State land, and State land occupied by the Authority for the purpose of constructing the Works, in accordance with any landscaping agreement reached between the Authority and the landowner.

12.2.3 For State land within the road reserve, both landscaping and turfing are permissible. In the case of State land outside of the road reserve, landscaping is permissible if NParks agrees to take over and maintain the completed landscaping.

12.2.4 Where a fully automated irrigation system is specified it shall be provided with a central control system, rain sensor and water level monitoring system. Details of such automated irrigation system are to be submitted to NParks for approval.

12.3 HANDRAILS AND RAILINGS

12.3.1 Handrails shall be designed for loadings in accordance with the requirements of Chapter 3 of this specification.

12.3.2 Standard safety railing shall be provided for all open drains more than 1m deep. The railings shall be in accordance with the standard specified in the Public Utilities Board (PUB)'s Code of Practice on Surface Water Drainage.

12.3.3 Where railings are required in areas other than the station premises for public safety, they shall be provided in accordance with the latest LTA Standard Railings Details found in the Standard Details of Road Elements.

CHAPTER 13

E&M INTERFACE

13.1 GENERAL

This chapter covers particular E&M requirements that are to be incorporated into the C&S design.

13.2 ELECTRICAL SUBSTATION

Electrical substations are required within the stations and as independent structures within the depot.

13.2.1 Cable Chamber

13.2.1.1 A cable chamber shall be provided beneath the full area of the electrical equipment rooms. The chamber shall be a through void as far as practicable and treated as one fire compartment. The cable chamber shall have a height of at least 1700 mm.

13.2.1.2 The cable chamber should be dry and free of water. Cable entry into the cable chamber through external ducts shall be sealed by means of a multiple cable transit (MCT). The cable chamber shall be waterproofed.

13.2.1.3 The cable chamber floor shall slope towards one end and floor wastes shall be provided for draining of any water present in the cable chamber and connected to the surface water drainage system.

13.2.1.4 Where it is not possible to construct main access door(s) to the cable chamber at least two manholes of 750mm x 750mm size shall be provided. These manholes shall be complete with a galvanised steel ladder. The manhole cover shall be light enough for one man to handle and designed to withstand 5kN/m^2 live load. These manholes shall be located at the two ends of the equipment room, flush to the floor, free from any obstruction and away from the escape route.

13.2.2 Others

13.2.2.1 Fixed pulling points (eyebolts) of appropriate tonnage shall be provided in the transformer rooms for installation and withdrawal of the transformers. The eyebolts must be recessed to the wall, soffit or floor. Those in the floor shall be provided with a cover and fitted flush to the floor surface. Each eyebolt shall be tested to 1.5 times safe working load and certified by Approved Person.

13.3 PLATFORM TOUCH VOLTAGE PROTECTION

13.3.1 General

13.3.1.1 The Contractor shall design and detail an effective platform insulation system to prevent passengers on the platform from possible electric shocks caused by touch voltage when boarding/alighting or touching the train or when touching the Platform Screen Doors (PSDs).

13.3.1.2 A "protection zone" (as defined below) by the platform edge areas adjacent to the tracks shall be electrically isolated (for example, from the station structure/electrical earth, or from traction earth, etc.) All appurtenances and the finishes to all structures (including floors, walls and columns) that fall within the protection zone shall be electrically isolated from earth, or provided with a suitable insulated coating, to avoid harmful touch potentials.

13.3.1.3 The extent of the protection zone shall be :

- Vertically, between the top of platform structural slab level and a minimum 2.5 metres above platform finished floor level.
- Transversely, from the platform edge to a minimum distance of 1.8 metres into the platform from any part of the inner face of the platform screen doors (PSDs) assembly, and from any metallic clad platform edge column,
- Longitudinally, for the full length of the PSD assembly to encompass anywhere that is within 1.8 metres of the PSD assembly. In situation where the end return door, in the swing open position is within the PSD protection zone, the protection zone shall be extended a further 1.8m beyond the end return door. See Figure 13.1 and 13.2.

13.3.1.4 The finishes to the platform between the platform edge and the remote side of the protection zone shall be fully electrically insulated from the structural slab below, and from all adjacent finishes and/or structures at the boundaries of the protection zone.

13.3.1.5 All cladding, including vitreous enamel, on walls or columns shall use electrically insulated fixings. Skirtings around walls or columns that fall within the protection zone shall also be isolated.

13.3.1.6 Insulated breaks in the finishes shall be provided at the boundaries of the protection zone to ensure that the isolated areas are not earthed to the non-isolated areas.

13.3.1.7 Metallic handrails that run along the platform edge in the buffer areas and which fall within the protection zone shall have an insulated finish.

13.3.2 Minimum Insulation Level

13.3.2.1 Notwithstanding 13.3.1 above, the minimum insulation levels shall be:-

a) A minimum platform floor to earth resistance of 10,000 and 35,000 ohm over a 300 x 300 mm area at 250V DC under damp and dry conditions respectively.

b) A metallic finishes (e.g. handrails, metallic cladding, etc.) to earth resistance of 50,000 ohm at 500V DC under damp condition.

c) A minimum resistance of 0.25M-ohm between PSD and Earth.

13.3.3 Insulation Details

No embedded conduit, trunking or service pipe shall be allowed to run in or through the insulated areas.

No metal dividing strip shall be allowed on the platform finishes within insulated areas.

Expansion joints on platform shall be kept to a minimum. Expansion joints shall not allow passage of water or moisture.

13.4 WATER AND ELECTRICAL EQUIPMENT

13.4.1 General Protection

13.4.1.1 No water pipe, whether gravity fed or under mains pressure, shall be located within electrical rooms (substations, switch rooms, signal rooms, battery rooms, CER, etc) or associated cable chambers. Such prohibited pipes include but are not limited to : -

i) Rain water down pipes;

ii) Sewerage & sanitary plumbing pipes including local penetrations through ceiling slab;

iii) PUB water mains supply;

iv) Internal water services plumbing reticulation;

v) Sprinkler and dry riser mains;

vi) Drainage pipework.

vii) Pumped discharge pipework for drainage and sewerage & sanitary plumbing.

13.4.1.2 No floor trap is to be cast into the slab directly above an electrical room.

13.4.1.3 No "wet" (ECS plant room, sprinkler tank room, toilets, cleaners room etc) rooms shall be located above electrical plant rooms.

13.4.2 External Cable Manholes and Cable Ducts

13.4.2.1 External cable manholes shall be positively drained, and shall comply with Section 11.2.3.9.

13.4.2.2 Where cable ducts enter an electrical room or cable chamber, the duct shall slope away to minimise seepage into the room or chamber.

13.5 E&M EQUIPMENT DELIVERY ROUTES

13.5.1 The design shall provide adequate routes for E&M equipment to be brought into the stations and the depot and for subsequent replacement. These routes shall be clearly marked on a separate set of drawings, which shall also show the information to be agreed as below

13.5.2 The routes shall be subject to co-ordination between the Civil Contractor and the Systemwide Contractors who shall agree for the following:-

- the mode and location of supply of equipment to the site;
- the method of unloading;
- the route of the equipment from point of unloading to final position;
- the lifting requirements throughout the route and method of transport;
- the weight of the equipment and its effect on the design loads for the structure, capacity of lifting hooks, lifting beams etc, either temporary or permanent;
- requirements for temporary decking, and staging;
- protection required for equipment and structure/finishes.

13.5.3 It is envisaged that some equipment will be lowered from ground level to concourse level and platform level via the draught relief shafts. For such cases, the design of vent shafts shall take into consideration the provision of demountable louvres, panels, etc. For Civil Defence stations, staging may be necessary in the bomb pits to facilitate the movement of equipment. Such staging shall be designed to support a static load of 20 tonnes, or such higher load as may be required by the Systemwide Contractors. For delivery from tunnels, floor access hatches above the trainway with lifting beams shall be provided. The lifting beam capacity shall be the subject of co-ordination between Civil Contractor and Systemwide Contractor.

13.5.4 Fixed lifting points (eyebolts) shall be provided for installation and replacement of major pieces of equipment. The minimum requirements are:-

- For Traction Substation Transformers

  3 Nos. 5 tonne safe working load capacity lifting point in soffit of the structure(s) above the transformer, located in front of the intended location of the transformer.

- For Service Substation Transformers

  1 no. 5 tonne safe working load capacity lifting point in soffit of the structure(s) above the transformer, located in front of the intended location of the transformer.

- For Escalators

  8 nos. 5 tonne safe working load capacity lifting points for each escalator in the soffit of the structure(s) above the escalator. The safe working load capacity shall include the lateral loading at an inclination of 60 degree from vertical. Additional lifting points shall be provided when the escalator's rise exceeds 8m.

- For Lifts

  2 nos. 3 tonne safe working load capacity lifting points or 1 no. 3 tonne safe working load capacity lifting beam for each lift at each lift shaft and each lift motor room

- For Generator

  1 no. 6 tonne safe working load capacity lifting point for each generator.

  In all cases, location and safe working load shall be co-ordinated with the Systemwide Contractor.

13.6 ELECTRICITY SUPPLY TO CIVIL EQUIPMENT

13.6.1 Various pieces of equipment supplied by the Civil Contractor will require an electrical supply to operate. The design shall be co-ordinated with the E&M Contractor regarding the location, power requirements, cable routing, termination and connection for power supply to all such equipment.

13.6.2 The equipment supplied by the Civil Contractor requiring power supply includes (but may not be limited to): -

- Drainage Pumps
- Sewerage Ejectors
- Motors for Sliding Doors
- U.V Filters
- Hand Dryers
- Toilet Sensors
- Motorised Roller Shutters
- Automatic Station Entrance Doors

13.6.3 The design shall allow for a 400/230V supply with a $\pm$10% voltage and $\pm$2% frequency variance at the normal and emergency main low voltage distribution boards. The design shall be co-ordinated with the Electrical Services (including UPS) Contractor(s) with respect to the voltage drop between the main LV boards and the input terminals of their equipment, to ensure that the equipment will operate over the full range of supply conditions identified above. The design shall incorporate reduced voltage starting where necessary to ensure the tolerances as “seen” by the equipment are not exceeded.

13.7 EARTHING SYSTEM

13.7.1 General

a) The basic design for the earthing system is indicated on the Authority’s Earthing System Interface Drawing.

b) The Civil Contractor shall carry out individual soil resistivity test at each end of the station prior to construction of the station base slab.

The soil resistivity test shall use the Wenner 4-pin method and the results submitted by the Civil Contractor shall be endorsed by a Registered Electrical Professional Engineer (PE). The Civil Contractor shall select a minimum of 2 test locations for each station subject to the approval of the Engineer. Five sets of tests shall be conducted at each location; each set at pin spacing of 2m, 4m, 6m, 8m and 10m respectively.

c) The design, installation and testing of the earthing system shall be in accordance with SS CP16. The passing criteria for earthing resistance shall be less than 1Ω. For Rapid Transit Systems, the earth mat shall be designed in accordance with ANSI IEEE 80.

d) The Civil Contractor shall prepare the necessary detailed design calculations, working drawings and test procedures and submit to the Engineer for approval.

13.7.2 Earthing Mat Design Requirements

a) The earthing mat shall be designed to limit the coupling between the lightning protection and the power system earth mat to 110V when a discharge of 100KA from a lightning strike occurs. The coupling device connecting the lightning protection and power system earthing mats shall be under the scope of the Civil Contractor. The Civil Contractor shall co-ordinate with the Electrical Services Contractor to ensure that the design meets the above design criteria.

13.7.3 Installation and Execution

a) The ringed earthing mat shall comprise earth rods inclusive of $95mm^2$ bare stranded copper wire laid 300mm below basement slab/ground level. The selection of electrode material shall be resistant to corrosion in the type of soil in which it will be used. Earth rods shall be coupled together with silicon aluminium bronze coupling and the copper wire joints shall be by exothermic weld. All exothermic welds shall be inspected by the Engineer before backfilling.

b) Earth riser cables $185mm^2$ cross linked polyethylene (XLPE) shall be brought from the earth mat up through the basement floor or wall to each of the equipment rooms and two nos 95mm insulated earth cables from the earthing mat to the two test locations located aboveground at station air shafts, as shown on Authority's Drawing.

c) At each riser cable entry through the basement floor or wall, a heavy-duty pipe sleeve complete with water stop seal shall be provided to prevent the ingress of water. Details shall be submitted for the Engineer's approval.

d) The Civil Contractor shall co-ordinate with the Power Supply Equipment and Cabling Contractor for termination of the earth riser cables onto the main earth bars in accordance with the Authority's Earth System Interface Drawing.

e) The copper electrodes at the earth inspection chamber at ground level shall have a label "Electrical Earth - Do Not Remove".

13.7.4 Testing

a) The Civil Contractor shall carry out the following test:

   i) Soil resistivity measurement

   ii) Earth mat resistance measurement. Measurement to be endorsed by Civil Contractor’s Electrical Professional Engineer.

   All test equipment shall be calibrated by SPRING Singapore or an accredited laboratory.

b) The Civil Contractor shall also carry out interface co-ordination with the relevant Interface or Systemwide Contractors for joint witnessing of all earthing tests.

c) The Civil Contractor shall carry out the following earthing mat tests:

   i) Individual earthing mat test at each end of station

   ii) Combined earthing mat test

   iii) Continuity test after earth risers are terminated on earth bars, and

   iv) Final earthing mat test as indicated below in 13.7.4e.

d) The Contractors shall submit plots of each resistance curve together with all test results to the Engineer for approval.

e) Four weeks prior to 22 kV power-on of MRT station power supplies, the Civil Contractor’s Electrical PE shall carry out a final earth test. It is essential that all inspection chambers be properly completed before the final earth test.

f) The Civil Contractor’s Electrical PE must submit 6 original copies of endorsed earthing certificate (Declaration of the Earthing System), test report/results, earthing mat design calculations and as-built drawings to the Engineer prior to the commissioning of station power supplies.

13.8 CABLE AND PIPE DUCTS

13.8.1 The position and requirements of all ducts shall be agreed in co-ordination with the Systemwide Contractors.

13.8.2 The ends of cable ducts shall be provided with bell-mouths to avoid possible damage to the cable.

13.8.3 UPVC shall not be used within the stations or tunnels and the Contractor shall specify an alternative material (example, ABS, FRE or approved equivalent) for cable ducts, sleeves, etc.

13.8.4 Spare ducts shall be provided for future use. The numbers (minimum 20% rounded up to the nearest whole number) and locations shall be agreed in co-ordination with the Systemwide Contractors.

13.8.5 Cable and pipe ducts shall be designed to allow maintenance of the services installed inside.

13.9 EQUIPOTENTIAL BONDING

13.9.1 The Civil Contractor and all Systemwide Contractors shall provide the necessary Equipotential Bonding (EPB) to meet all requirements of the Singapore Standard, CP5.

13.9.2 Items requiring EPB and their sizing requirements are detailed in Table 1 and 2 respectively.

13.9.3 Where EPB provisions are identified to be required in Table 1, the Civil Contractor shall provide the supplementary bonding to the nearest EPB conductor.

13.10 CABLE BRACKETS AND OTHER E&M FIXINGS IN TUNNELS

13.10.1 The position of all cable brackets and other attachments to the tunnel wall, provided or designed by another party, shall be agreed in co-ordination with that party.

13.10.2 Cable brackets and the like shall be aligned at right angles to the gradient of the tunnel.

13.10.3 In segmental concrete lined tunnels, the requirements of Clause 7.5.5 shall apply. Each cable bracket or other attachment shall be provided with sufficient adjustment (slotted holes or similar) or be provided with secondary fitments to ensure that the requirements of Clause 7.5.5 are upheld.

TABLE 1: SUMMARY OF EPB REQUIREMENTS

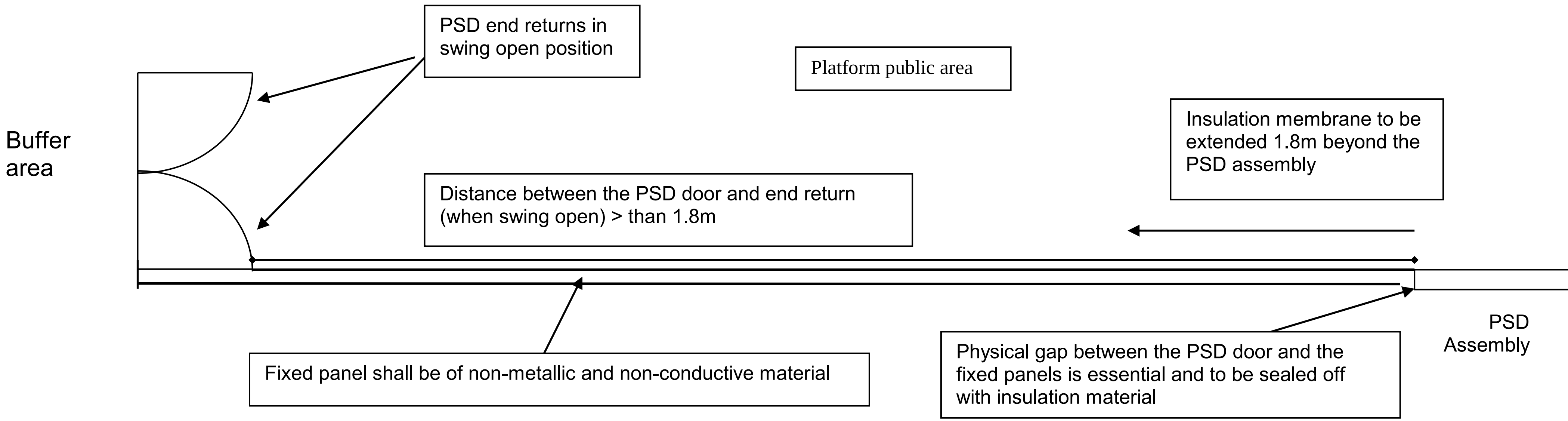
	Metallic Part	Comments
1	All incoming and outgoing service pipes such as water, fuel, dry riser, pumping main	Requires EPB. To be connected by a main EPB conductor to the earthing terminal as stipulated in CP5:1998, Cl. 413.2. See Figure 1. Pipe joints to be checked for electrical continuity otherwise bonding required. Pipes along the trainway to be insulated from its steel brackets to prevent leakage paths for stray currents.
2	Metal tanks	Requires EPB. May be connected electrically to incoming pipe. See Figure 2.
3	All metallic cat-walks, platforms, hand-rails, staircases, ladders within 1.8m reach of pipes, tanks, cable trays cable ladders, trunking etc which have EPB.	Requires EPB with supplementary EPB conductor connected to pipes, tanks etc. See Figure 2.
4	Any metallic catwalks, platforms, handrails, staircases, ladders etc with attached electrical cabling or fittings.	Require EPB with supplementary EPB conductor connected to exposed conductive part of fitting.
5	Any metallic catwalk, platform, handrail, staircase, ladder etc, without electrical cabling or fittings and greater than 1.8m from pipes, tanks etc.	No EPB required.
6	Metallic door frames/doors controlled by electromechanical locking mechanism	EPB required with supplementary EPB conductor connected to exposed conductive part of fitting only where voltage rating of lockset exceeds 50V.
7	Metallic supports to electrically operated equipment without direct electrical contact with the equipment	EPB required with supplementary EPB conductor connected to exposed conductive part (related electrical equipment). See Figure 3.
8	Electrically operated roller shutters	EPB required with supplementary EPB conductor connected to exposed conductive part (casing of roller shutter motor).
9	Metallic wall cladding (excluding VE)	EPB limited to panels and framework components containing, or immediately adjacent to, electrical socket outlet or other sources of electricity. Supplementary EPB conductor

		connected to exposed conductive part of fitting. See Figure 4. No EPB required for VE panelling. Details of wall cladding to be examined to ensure electrical continuity throughout.
10	Common trunking provided by Civil Contractor	EPB required with supplementary EPB conductor connected to nearest main EPB conductor.
11	Raised floor system	EPB required with supplementary EPB conductor connected to nearest earthed part (e.g., cable tray). Details of floor system to be examined to ensure electrical continuity between panels and between panels and supports.
12	Electrical facilities in toilets and shower rooms (e.g. hand-dryer, water heater, extract fan etc)	No EPB required.
13	Ceiling system	No EPB required.
14	Blast doors	No EPB required.
15	Exposed metallic parts of building structure, including roof trusses.	EPB required (as Cl 413,2 of CP5). To be connected to earthing terminal. However for roof trusses, may be connected to lightning conductor earth and no further EPB is required. Electrical continuity of structure to be checked.
16	Steel support beam to PSD	No EPB required.
17	Lifting beams and hooks	No EPB required
18	Framework of PSC and fixed metallic furniture within.	EPB required. To be connected to nearest earthed part (eg cable tray).

Table 2: Sizing Requirements as per CP5: 1998, Section 547

Type	Size	Max	Min
Main EPB	½ Earthing Conductor	25 mm^2	6 mm^2
Supplementary EPB	½ Circuit Protective Conductor (cpc)	-	2.5 mm^2

Figure 13.1 - Distance between End Return and PSD > 1.8m



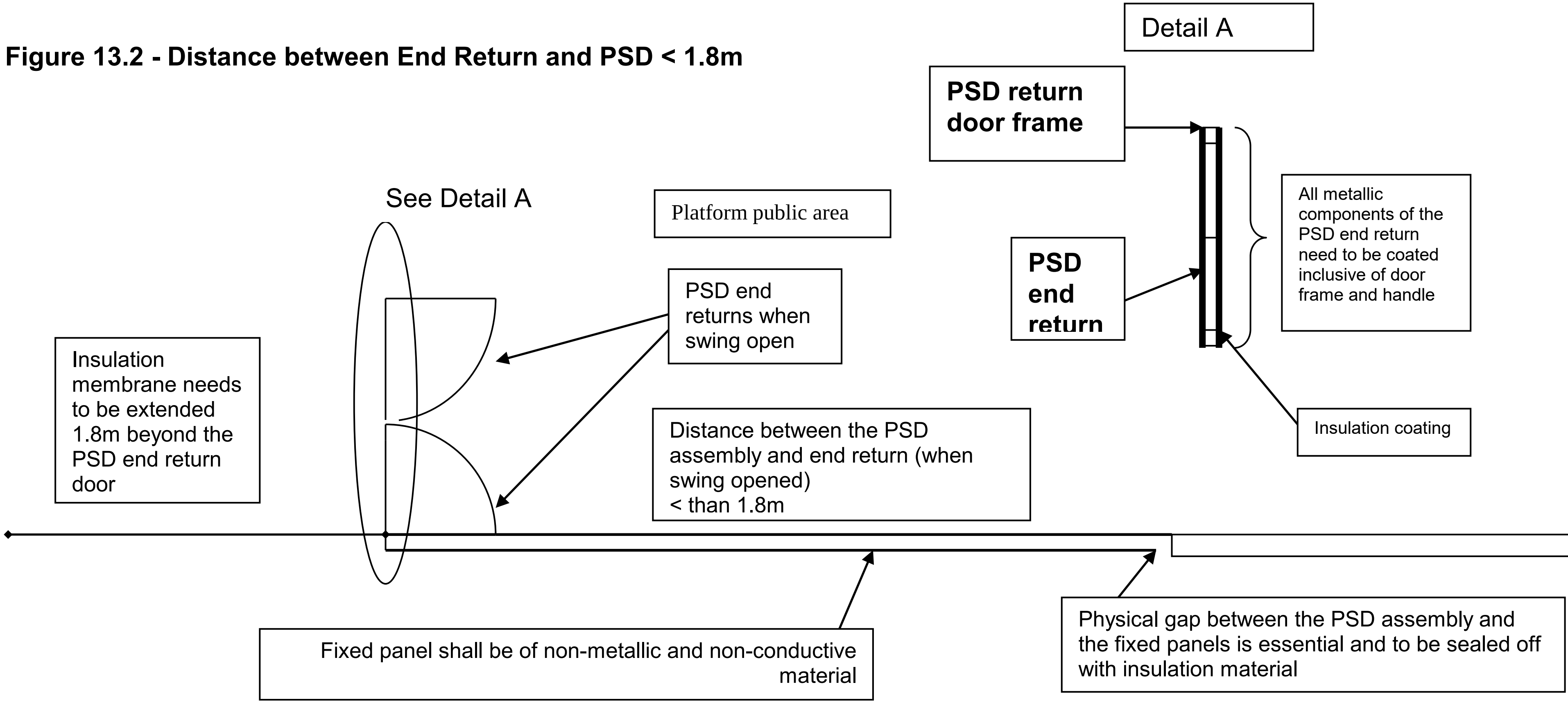
1) When end return door in open position and the distance between the end return and the PSD is more than 1.8m, the following are acceptable

   a) No coating to PSD end turn is required

   b) End return not necessary to be ‘electrically floated’

   c) Insulation membrane needs to be extended 1.8m beyond the PSD assembly

Figure 13.2 - Distance between End Return and PSD < 1.8m



2) When end return door in open position and the distance between the end return and the PSD is less than 1.8m, the following are required

a) Coating to PSD end turn is required. (All metallic components of the PSD end return need to be coated with insulation material. Refer to Detail A above)

b) End return need to be ‘electrically floated’

c) Insulation membrane needs to be extended 1.8m beyond the PSD end return door.

CHAPTER 14

STRAY CURRENT CORROSION CONTROL FOR RAILWAYS

14.1 INTRODUCTION

14.1.1 General

14.1.1.1 The Stray Current Corrosion Control (SCCC) system for steel wheel on steel rail Rapid Transit Systems (RTS) with d.c. traction supply shall have the following design objectives:

(a) Provide a SCCC system that will ensure a 120-year design life for railway structures.

(b) Provide facilities to monitor and mitigate the stray current effects on the RTS facilities and structures.

14.1.1.2 The criteria given below shall be used as a guide only. Each contractor shall evaluate the requirements for SCCC within his design, coordinate all relevant interface details with all necessary parties, and shall produce detailed design and drawings accordingly

14.1.1.3 The trackwork contractor shall take the lead in coordinating and producing an interface SCCC diagram for the entire line for coordination with all parties.

14.1.2 General Requirement

14.1.2.1 The SCCC system shall generally include the following:

(a) Isolation and/or control of all possible stray current leakage paths to minimise stray current effect on RTS and adjacent structures, public utilities and RTS services.

(b) Detection and monitoring of stray currents which do occur.

(c) Stray current drainage (collection) system which, when put into use, provides a return path for the stray current back to the traction substation negative busbar via drainage diodes.

14.1.2.2 The amount of stray current that will leak out from the track and power system shall be assessed by the Electrical Services contractor. Based on this amount of leakage current, the Civil Contractor shall prepare a quantitative analysis of the corrosion effects on the transit structure and appropriate provisions made in the design to ensure that the objectives given in Clause 14.1.1.1 are achieved.

14.2 SYSTEM REQUIREMENTS

14.2.1 Trackwork

14.2.1.1 The size of the running rails shall be adequately sized to reduce voltage drop in the running rails, which in turn will reduce leakage of stray current

14.2.1.2 The minimum track to structure earth resistance under dry and damp conditions shall be 10 ohm-km before any equipment/cables are connected. Where the measurement is carried out in smaller section, the measured resistance shall be normalised accordingly. E.g. for 500m length of track, the measured resistance shall be greater than 20 ohm (ie. 10 ohm-km/0.5 km). The minimum track to structure earth resistance can be achieved by the following methods as appropriate:

(a) Insulating pad beneath rails
(b) Insulation of rail fastenings
(c) Insulating pad beneath base plate
(d) At level crossings and similar locations, rail shall be sealed by a non-conducting filler section (Fig. 14.11)

14.2.1.3 The bottom face of running rails shall be at least 50 mm above the in-situ track concrete or ballast.

14.2.1.4 Guard rails on viaduct tracks shall be fixed on insulated fastenings. Guard rails shall be insulated from the running rails and include 20mm air gaps at every 18 metres approximately.

14.2.1.5 Permanent IRJs (complete with jumper cables) shall be provided at locations close to traction substations to segregate sections of track into electrically independent lengths to facilitate future track to earth resistance measurement. Cables shall be adequately sized based on the worst case scenario for the traction return current.

14.2.2 Elevated MRT Stations and Viaducts (Fig. 14.5 & Fig. 14.8)

14.2.2.1 The control of leakage at source shall be by insulation of the running rails as described in Clause 14.2.1 above.

14.2.2.2 Protection of viaducts against stray current corrosion shall also be provided by a high electrical resistant waterproof membrane above the structural concrete and below the ballast. The membrane shall be protected by concrete screed.

14.2.2.3 A stray current collection mesh typically formed of welded steel mesh shall be embedded in the concrete screed protecting the waterproof membrane for every track. Each panel of mesh shall be spot welded to the next to give good electrical continuity. The mesh shall be electrically continuous over each viaduct span and over each crosshead (if applicable), but electrically separated from each other. The maximum electrical resistance of the mesh shall be less than 0.5 ohm per km.

14.2.2.4 A monitoring cable shall be welded to one end of each discrete mesh and connected to a drainage terminal box for stray current monitoring and drainage purposes.

14.2.2.5 A stray current drainage cable shall be installed along each trackway inter-connecting every drainage terminal box and terminating at the traction substation negative busbars via drainage panel.

14.2.2.6 Lightning earth electrodes shall not be used as reference electrodes for stray current monitoring. Instead a portable half-cell electrode shall be used.

14.2.3 Underground Structures (Fig. 14.6, Fig. 14.7 & Fig. 14.8)

14.2.3.1 The control of leakage at source shall be by insulation of running rails as described in Section 14.2.1 above.

14.2.3.2 In the second stage concrete under the track, a stray current collection mesh, typically formed of panels of welded steel mesh shall be installed. Each panel of mesh shall be spot welded to the next to give good electrical continuity over discrete lengths of 100m. At every 100m there shall be an electrical discontinuity created by leaving a gap of 100mm between successive lengths of mesh. The maximum electrical resistance of the mesh shall be less than 0.5 ohm per km.

14.2.3.3 A monitoring cable shall be welded to one end of each discrete mesh and connected to a drainage terminal box for stray current monitoring and drainage purposes.

14.2.3.4 A stray current drainage cable shall be installed along each trackway inter-connecting every drainage terminal box and terminating at the traction substation negative busbars via drainage panel.

14.2.4 At-Grade and Transition Sections (Fig.14.9)

14.2.4.1 The control of leakage at source shall be by insulation of running rails as described in Clause 14.2.1 above.

14.2.4.2 In ballasted track on transition structures between tunnel and viaduct, the stray current collection and monitoring system shall be of the form specified for viaducts in Clause 14.2.2 but with the welded steel mesh in lengths as specified for slab track.

14.2.4.3 For concreted track on transition structures between tunnel and viaduct, the stray current collection and monitoring system shall be of the form specified for underground structures in Clause 14.2.3.

14.2.4.4 No stray current drainage or monitoring provisions beyond that described in Clause 14.2.1 is required for ballasted at-grade sections of railway.

14.2.4.5 No utility pipe or service shall be located under the at-grade sections of railway.

14.2.5 Depots

14.2.5.1 The control of leakage at source shall be by insulation of running rails as described in Section 14.2.1 above as far as practicable.

14.2.5.2 The depot area should be electrically isolated from the main line by using permanent Insulated Rail Joints (IRJs) at appropriate locations. Remote control facilities should be provided to enable each IRJ to be bypassed to allow transfer of traction power to or from the depot in the event of substation power failure.

14.2.5.3 No monitoring or drainage system is required in the depot track area beyond that specified above.

14.3 SYSTEM COMPONENTS

14.3.1 Cabling

14.3.1.1 For underground stations and tunnels, low smoke, halogen free, fire retardant type cables with armouring shall be provided.

14.3.1.2 For above ground station and viaducts, fire retardant type cables with armouring shall be provided.

14.3.1.3 All cables shall have an insulation level of 1.8/3kV and be single core, multi-stranded copper conductor and XLPE insulated.

14.3.1.4 Cable sizes shall be as follows:

(a) Monitoring cable (from mesh to terminal box) – 35mm^2 copper

(b) Drainage cable (inter-connecting terminal boxes) – 185mm^2 copper

14.3.1.5 The location of the drainage cable shall be co-ordinated with the Electrical Services contractor.

14.3.2 Drainage Panels

14.3.2.1 A drainage panel shall be provided in each traction sub-station. It is used to provide a connection path between the drainage terminal box and the negative return box.

14.3.3 Drainage Terminal Boxes

14.3.3.1 Each drainage terminal box shall be provided with connection terminals for the monitoring cable and the drainage cable. Connections between terminals shall be provided within the box by tinned copper busbars or approved equivalent. The drainage terminal box shall be of stainless steel material and of IP65 construction with removable box cover.

14.3.3.2 Further details of the terminal box are shown on Figure 14.10.

14.3.4 Reference Electrodes

14.3.4.1 Zinc reference electrodes, two for each bound shall be installed in underground stations in the outer wall. Each zinc electrode shall be installed at each end of the station at convenient locations for connections to be made.

14.3.4.2 The zinc electrode interface terminal box (ITB), supplied by the Civil Contractor shall be of stainless steel material and of IP65 construction. The ITB shall be provided with removable box cover.

14.4 STRAY CURRENT LEAKAGE PATH CONTROL

14.4.1 General

14.4.1.1 No stray current leakage paths shall be formed between structural units, or between the structure and piped services, handrails and other metallic components located along the trainway. Dielectric insulation shall generally be provided to prevent stray current leakage.

14.4.1.2 No fixings or cabling for the trackwork shall affect the track insulation system or reduce substantially the track to earth/structure resistance.

14.4.2 Installations

14.4.2.1 Electrical insulation from the transit or depot structure is required for the following installations located along the trainway.

(a) Signalling equipment and/or their supports

(b) Platform Screen Doors

(c) Blue light station support frames and siding telephone support frames

(d) Metal pipes

(e) Lightning protection system to viaducts

(f) Earthing cables

(g) Sectionalising switches, high-speed circuit breakers and their supports

(h) Control boxes, test boxes, junction boxes etc and/or their supports.

14.4.3 Elevated Stations and Viaducts

14.4.3.1 The waterproofing and insulation membrane between ballast and structure concrete on the viaducts shall be a bonded system with minimum electrical resistivity of 5000×10^{11} Ω-cm and thickness not less than 2.5 mm.

14.4.3.2 Anchor bolts for viaduct bearings shall be isolated from the steel reinforcement in viaducts beams and cross-heads.

14.4.3.3 Any metallic handrail, fascia units, walkway and the like along the viaduct shall be electrically insulated from the steel reinforcement in viaduct beams and cross-heads.

14.4.3.4 Steel reinforcement and anchor bolts in concrete precast units used as fascia or similar shall not be in contact with the steel reinforcement of the viaduct beams.

14.4.3.5 The material of rainwater downpipes in columns shall be non-conductive.

14.4.3.6 Lightning down conductor (by others) shall be fixed with insulators to avoid electrical contact with steel reinforcement in cross-heads and columns.

14.4.3.7 An effective water drainage system shall be designed to prevent ponding of rainwater.

14.4.4 Underground Structures and Tunnels

14.4.4.1 Proper detailing shall be provided to prevent ponding of seepage water around track fastenings.

14.4.4.2 Handrails, walkways and other continuous metallic elements along the trainway shall be electrically insulated from the structure.

14.4.4.3 Tunnel segments shall be electrically separated from each other across all circumferential joints.

14.4.4.4 All non-railway metallic service pipes passing through or embedded within the RTS structure shall be insulated from the structure by a plastic sleeve.

14.4.4.5 Where zinc reference electrode is located, the structure rebar shall be extended as shown in figure 14.12. This is to allow the measurement of rebar potential with respect to zinc reference electrode be carried out to provide a basis for comparison of potentials in future. The probe shall be enclosed in a IP65 stainless steel box with removable cover and to be supplied by the Civil Contractor.

14.5 SYSTEM TESTING AND MONITORING (refer to Fig. 14.1 to Fig. 14.4 and Appendix 2)

14.5.1 Track to Structure Earth and Water Earth Resistance

14.5.1.1 The track resistance against current leakage shall be measured with respect to:

(a) structure earth by means of the terminals from the stray current collection mesh (steel mesh).

(b) water earth by means of the earth connection in the substation.

14.5.1.2 The track to structural earth resistance measurements shall be taken at the following two stages:

(a) For newly laid track before any power and signalling trackside equipment and cabling are connected.

(b) For integrated track system when power and signalling trackside equipment and cabling are connected.

14.5.1.3 As far as possible, different track support system shall be separated electrically for testing and monitoring:

(a) Main line shall be separated from the depot tracks.

(b) Tracks on viaducts shall be separated from those on at-grade sections, transition sections or inside tunnels.

14.5.2 Stray Voltage Level Monitoring

14.5.2.1 The stray voltage level monitoring shall involve measuring the steel mesh potential with respect to a portable half-cell electrode.

14.5.3 Substation Drainage Current Measurements

14.5.3.1 Provision shall be made to facilitate the following measurements:

(a) Traction current at output of rectifier dc traction feeders for each bound

(b) Negative busbar to earth voltage

(c) Drainage current distribution and total drainage current.

14.5.3.2 These measurements will be taken at traction substations to check the system balance and to provide a basis for comparison of potentials.

14.5.4 Other Tests

14.5.4.1 In addition to the above, the following tests shall be performed:

(a) Track to stray current collection mesh resistance measurements

(b) Electrical continuity test of steel mesh

(c) Commissioning tests for reference electrodes

(d) Insulation test for insulated installations.

14.5 5 Test Procedures

14.5.5.1 The types, methods and procedures of all tests and measurements including the format for recording the test results shall be submitted to the Engineer for acceptance.

14.5.5.2 Effective fault finding methods shall be incorporated and the technical specifications of instruments and equipment used to locate stray current leakage paths shall be specified.

14.5.5.3 All rail potential measurements as well as all traction and drainage current measurements shall be taken over a 24-hour period during trial running to enable both the traction peaks and non-traction natural potential to be observed and recorded.

14.5.5.4 All test instruments and measurement charts are to be provided by the commissioning party.

14.5.5.5 Upon completion of a commissioning test, a Test Inspection Certificate shall be endorsed and submitted by the Registered Professional Engineer of the relevant contractor to the Engineer for acceptance.

14.5.5.6 All test programmes shall, unless otherwise specified, be submitted to the Engineer at least four weeks before the tests start.

Appendix 1: Stray Current Corrosion Control Requirements - Installation

Serial no.	Items	Underground tunnel and station	Viaduct/ At Grade	Depot	Design, supply, installation and commissioning to be carried out by * (see footnotes at Appendix 2)
1	Welded steel mesh to be installed under the track. External connecting terminals to be provided.	✔	✔	NA	TW (underground) C (viaduct/ at grade)
2	Monitoring cable from steel mesh reinforcement to the drainage terminal boxes to be provided. (see Fig. 14.1 to Fig.14.10)	✔	✔	NA	TW (underground) C (viaduct/ at grade)
3	Drainage terminal boxes and drainage cables interconnecting drainage terminal boxes to be installed along tunnel wall or viaducts. (see Fig. 14.1 to Fig 14.10)	✔	✔	NA	ES (Drainage Cable) TW (Monitoring Cable and Drainage Box)
4	Drainage cable from appropriate drainage terminals inside drainage terminal boxes to the drainage panel in the traction substation (see Fig. 14.1 to Fig 14.4)	✔	✔	NA	ES
5	Installation of stray current drainage units and all associated accessories at traction substations.	✔	✔	NA	ES
6	Installation of drainage cable supports	✔	✔	NA	TW (underground) C (viaduct/ at grade)
7	Zinc reference electrodes	✔ (at stations only)	NA	NA	C
8	Provision of insulating sleeves for all non-railway metallic service pipes entering or penetrating the railway structure	✔	✔	✔	C
9	Running rails to be insulated by the following methods as appropriate: a) Insulating pad beneath rails b) Insulation of rail fastenings c) Insulation beneath base plate d) At level crossings and similar locations, rail shall be sealed by a non-conducting filler section.	✔	✔	✔	TW

Serial no.	Items	Underground tunnel and station	Viaduct/ At Grade	Depot	Design, supply, installation and commissioning to be carried out by * (see footnotes at Appendix 2)
10	Bottom face of running rail shall be at least 50mm above the in-situ track concrete or ballast.	✔	✔	✔	TW
11	Depot electrically isolated from the main line system by using: a) permanent IRJs (Insulated rail joints) at appropriate locations on reception tracks b) third rail sectionalising by unbridgable gap adjacent to IRJs for segregation c) third rail and running rail sectionalising switches and associated works with remote control facilities.	NA	NA	✔	a) TW b) TW c) ES
12	Lightning down conductor strip to be fixed with appropriate insulator.	NA	✔	NA	ES
13	The following installation shall be insulated from the transit structure or depot structure: a) Signalling equipment and/or their supports b) Platform screen doors c) Blue light station and siding telephone supports d) Dry riser pipe e) Pumping mains, drainage pipes, and other piped services provided by the civil contractor f) Earthing cables g) Sectionalising switches, circuit breakers, control/ test/ junction boxes, etc	✔ ✔ ✔ ✔ ✔ ✔	✔ NA ✔ ✔ ✔ ✔	✔ NA ✔ ✔ ✔ ✔	SE SE CS, ES MS C ES, C, TW, SE, ES

Appendix 2: Stray Current Corrosion Control Requirements – Testing and Commissioning

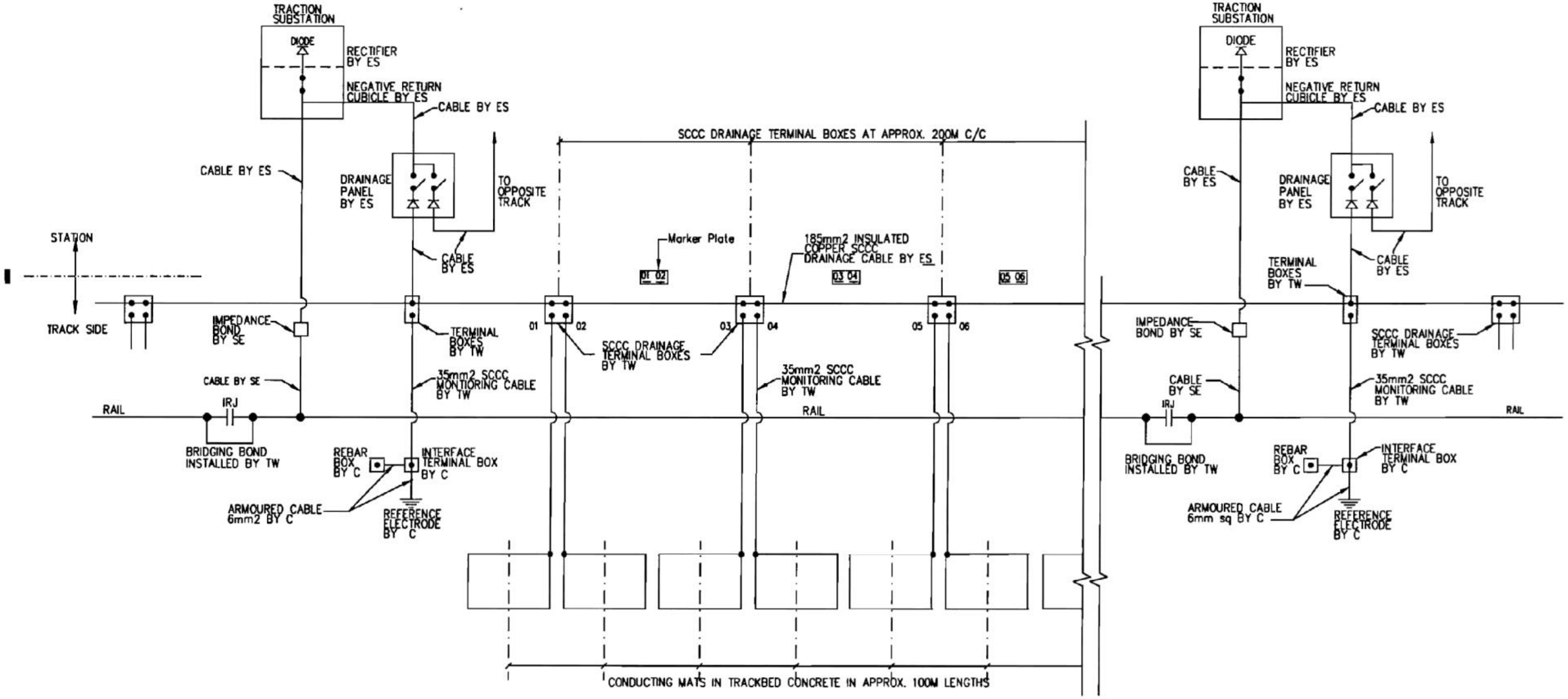
Serial no.	Commissioning Tests	Commissioning Test to be carried out by * (see footnotes)	Remarks
1	Commissioning test for zinc reference electrodes	C	At stations only.
2	Electrical continuity test of welded reinforcement mesh (Electrical resistance shall not be more than 0.5 Ω-km)	TW (underground) C (viaduct/ at grade)	Measurements to be taken before concreting.
3	Track to structure earth resistance measurements in accordance with EN 50122-2 shall be carried out before E&M installation. (Eg. Signalling, point machines, etc.) (Pass Criterion shall not be less than 10 Ω-km)	TW	Measurements to be taken when the tracks are newly laid and before the installation of trackside equipment and cabling. (Refer to Table - Integrated System Electrical Test Requirements at trackwork interface in the General Specification)
4	Track to structure earth resistance measurements in accordance with EN 50122-2 shall be carried out after E&M installation is completed. (Pass Criterion shall not be less than 7.5 Ω-km)	TW ES, SE	Measurements to be taken after the installation of trackside equipment and cabling. TW shall be responsible for carrying out measurement only. (Refer to Table - Integrated System Electrical Test Requirements at trackwork interface of the General Specification) ES, SE shall assist TW to conduct measurements. ES, SE shall be responsible for all site preparation including cleaning, disconnection and reconnection of crossbonding cables for TW to carry out the measurement.
5	Test on all insulation for stray current corrosion control purposes under Appendix 1, Item 13 (Pass Criterion: Electrical insulation resistance shall not be less than 10 kΩ at test voltage of 250V d.c. For minimum insulation level of PSD, please refer to DC chapter 13.)	C, SE, ES	Individual contractors are required to carry out and furnish test records to confirm that the respective installations have adequate electrical insulation resistance values.

Serial no.	Commissioning Tests	Commissioning Test to be carried out by *	Remarks
6	Stray current measurement in traction substation: a) traction current at dc traction feeder for each bound b) negative busbar to earth voltage (rail potential) c) drainage current	ES	24-hr measurement to be carried out during trial running. See Clause 14.5.5.3
7	Stray voltage level measurement with respect to reference electrodes/half cell electrodes for steel mesh for stray current collection mesh (via monitoring cable)	ES	24-hr measurement shall be carried out during trial running, see Clause 14.5.5.3. Measurement results are for record purposes only. Extent of testing to be determined by the Engineer.

<u>*Footnotes</u>

*** Legend to Appendix 1 and 2:**

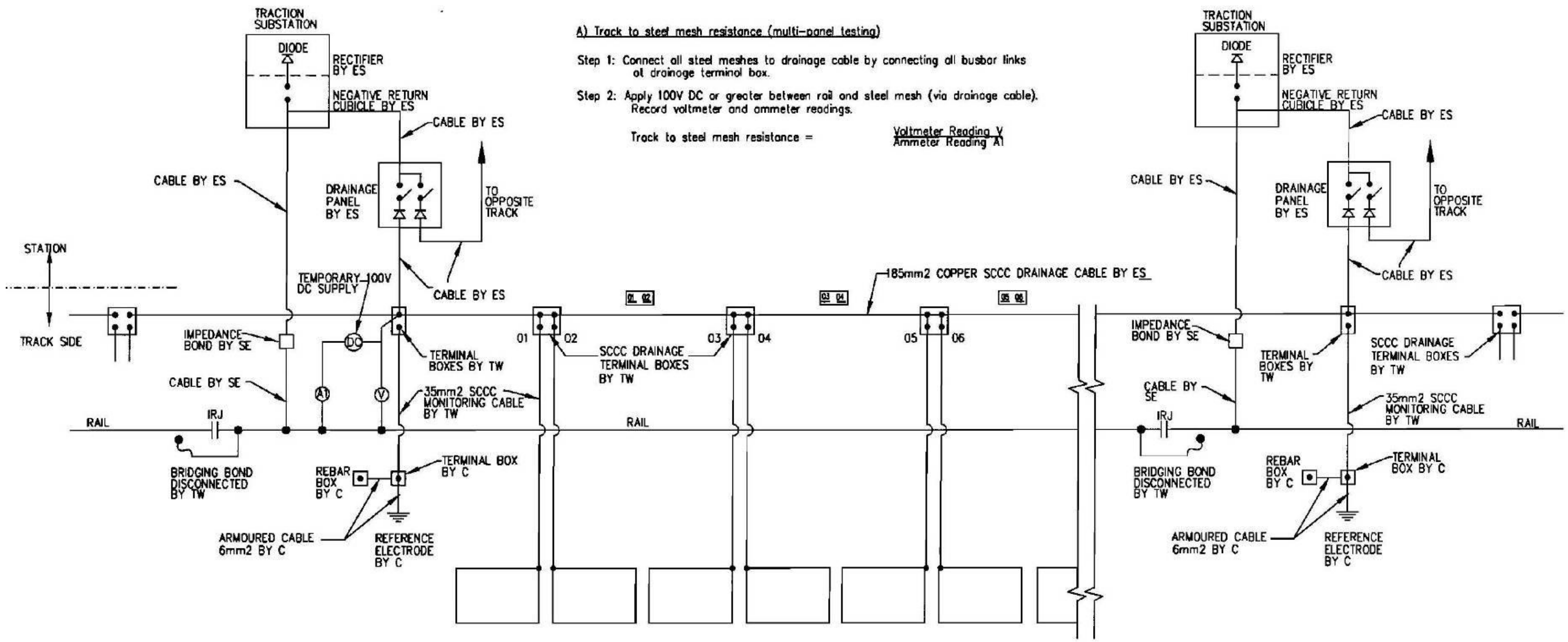
NA = Not Applicable
C = Civil contractor
TW = Trackwork contractor
SE = Signalling Equipment and Platform Screen Door contractor
ES = Electrical Services contractor (inclusive of Power Supply)
MS = Mechanical Services contractor
CS = Communication System contractor



Note:
C :Civil contractor
TW :Trackwork contractor
SE :Signalling Equipment & Platform Screen Door contractor
ES :Electrical Services contractor
CS :Communication Services contractor

FIG. 14.1 – CIRCUIT DIAGRAM (NORMAL OPERATION)

STRAY CURRENT CORROSION CONTROL SYSTEM CONCEPT



Note:
C : Civil contractor
TW : Trackwork contractor
SE : Signalling Equipment & Platform Screen Door contractor
ES : Electrical Services contractor
CS : Communication Services contractor

FIG. 14.2 – TESTING MODE (1) (MULTI-PANEL TESTING)

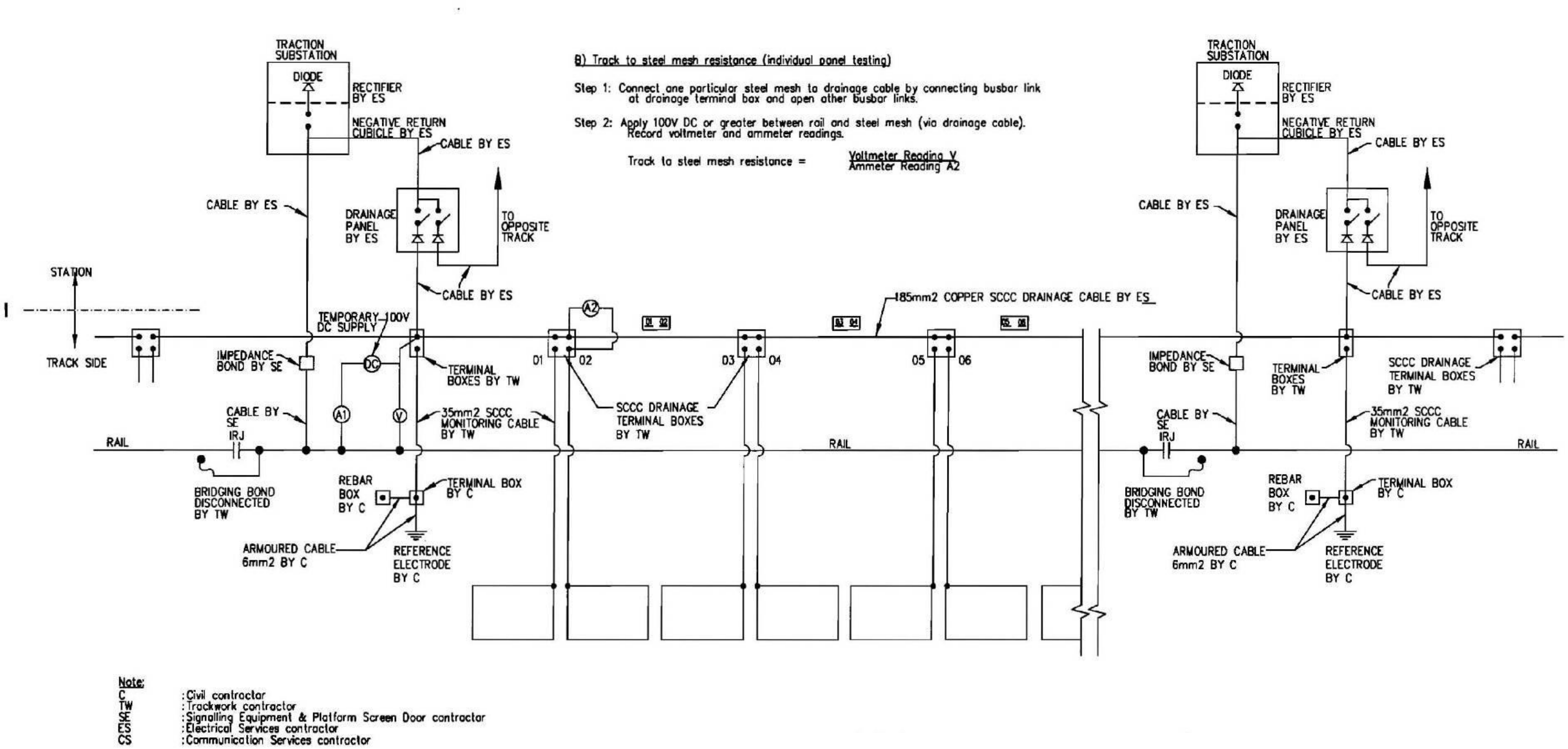


FIG. 14.3 – TESTING MODE (2) (INDIVIDUAL PANEL TESTING)

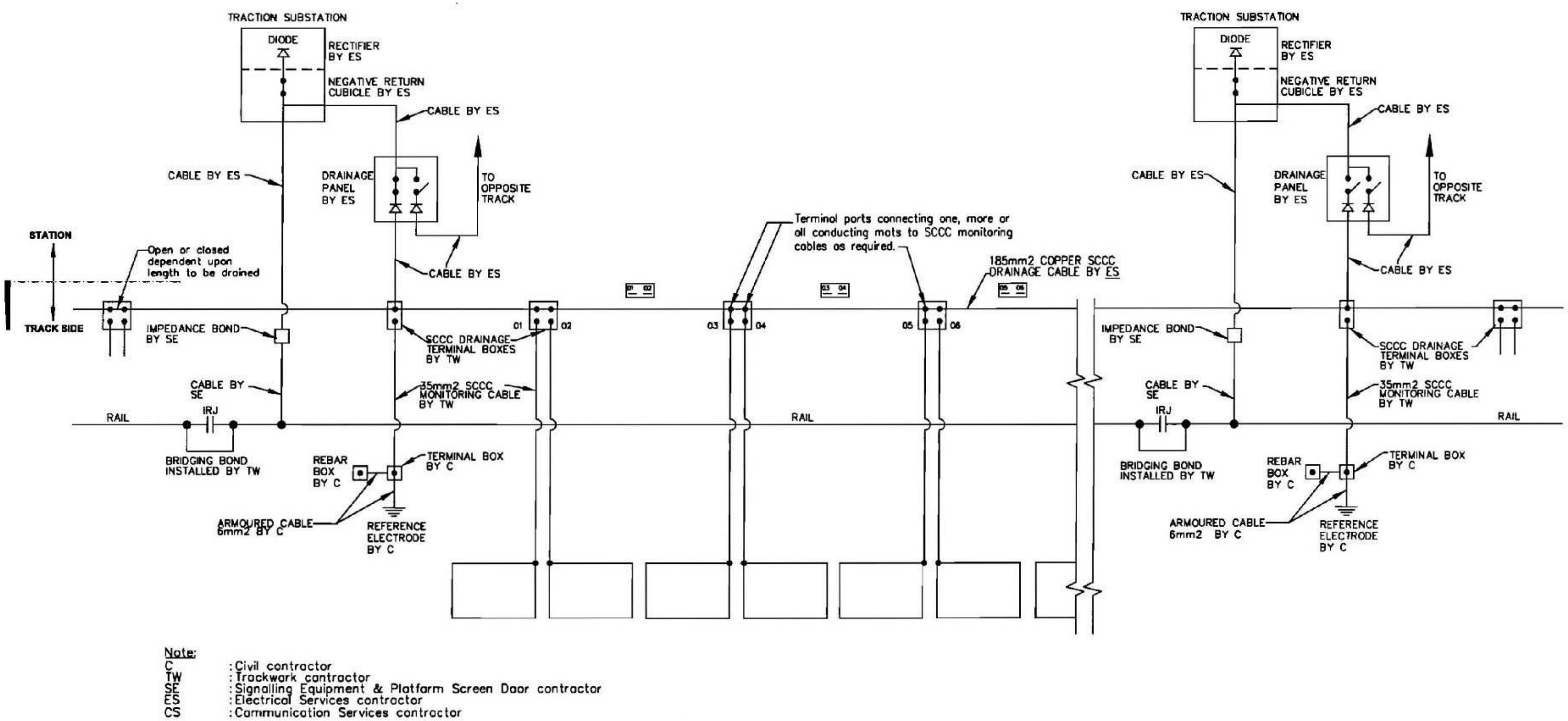


FIG. 14.4 – DRAINAGE MODE

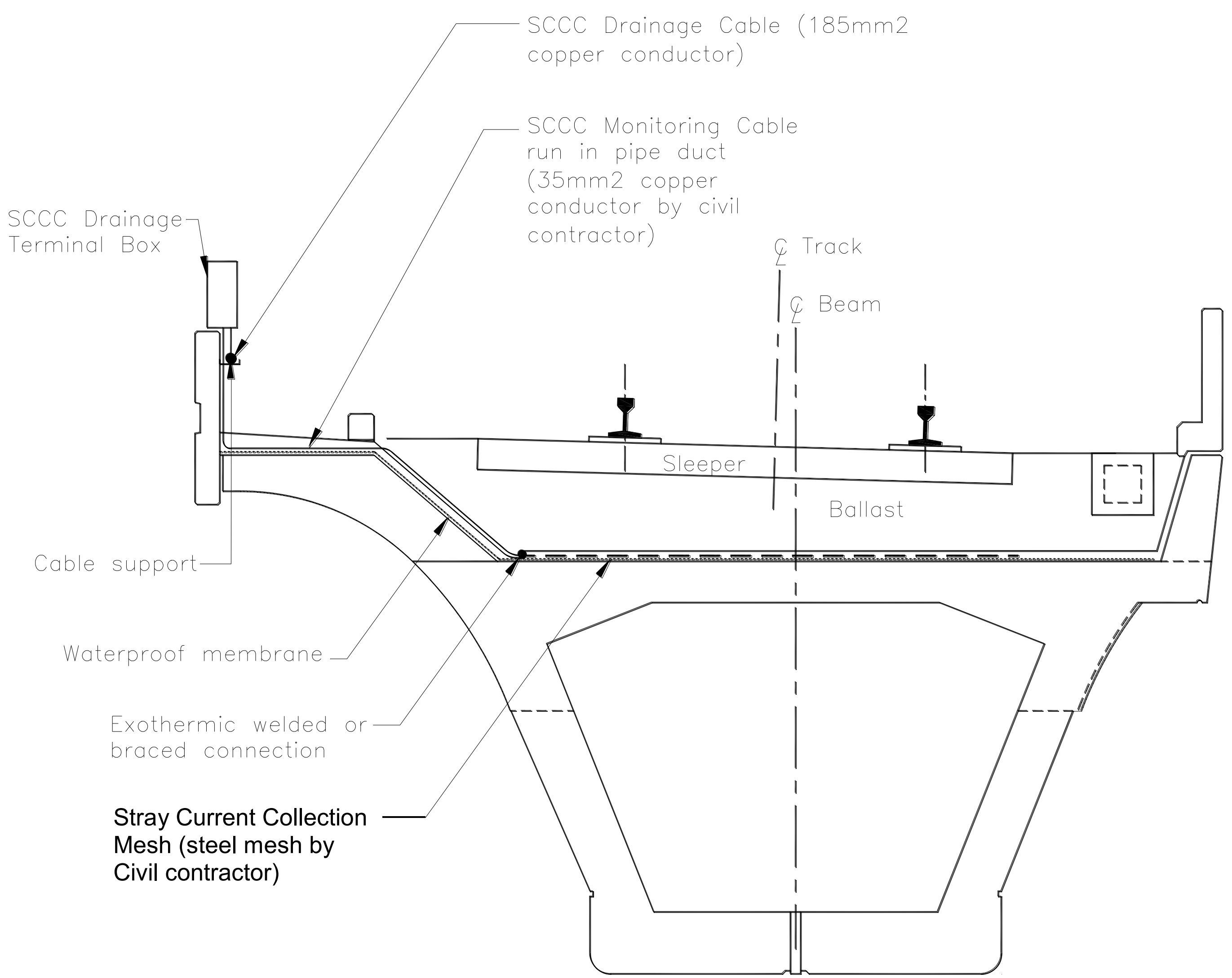


FIG. 14.5 – SECTION THROUGH VIADUCT

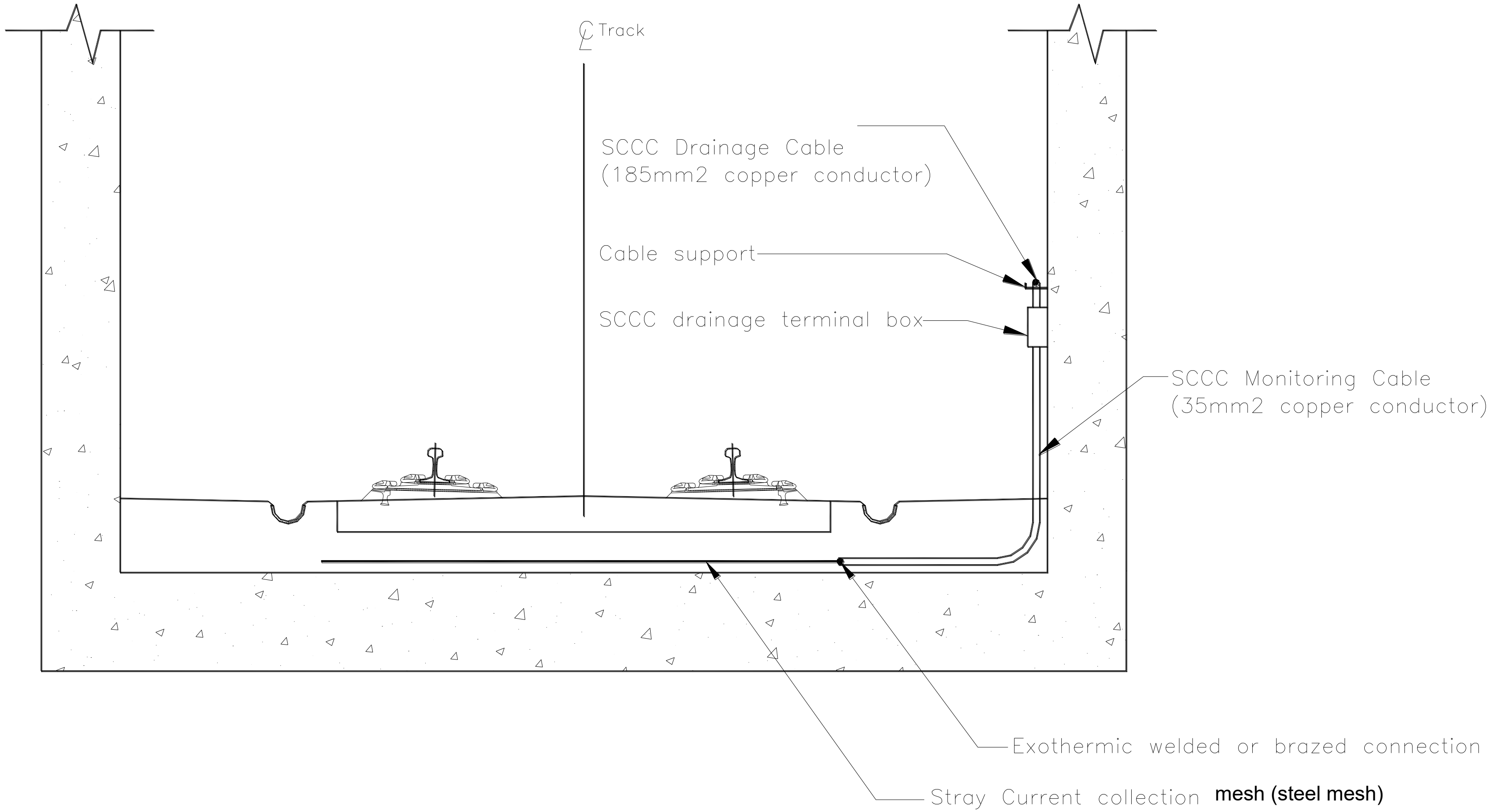


FIG. 14.6 – SECTION IN CUT AND COVER TUNNEL

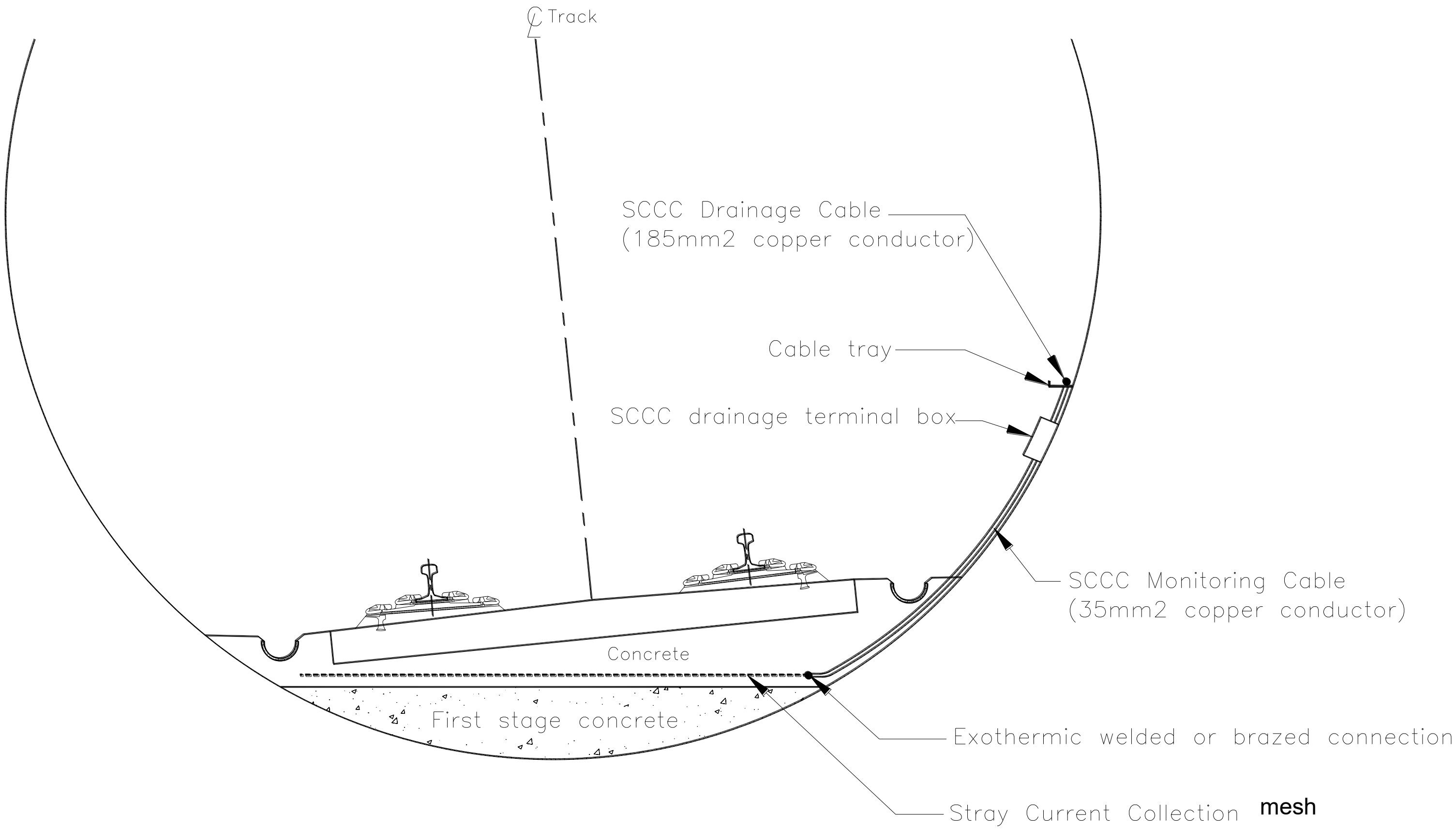


FIG. 14.7 – SECTION IN BORED TUNNEL

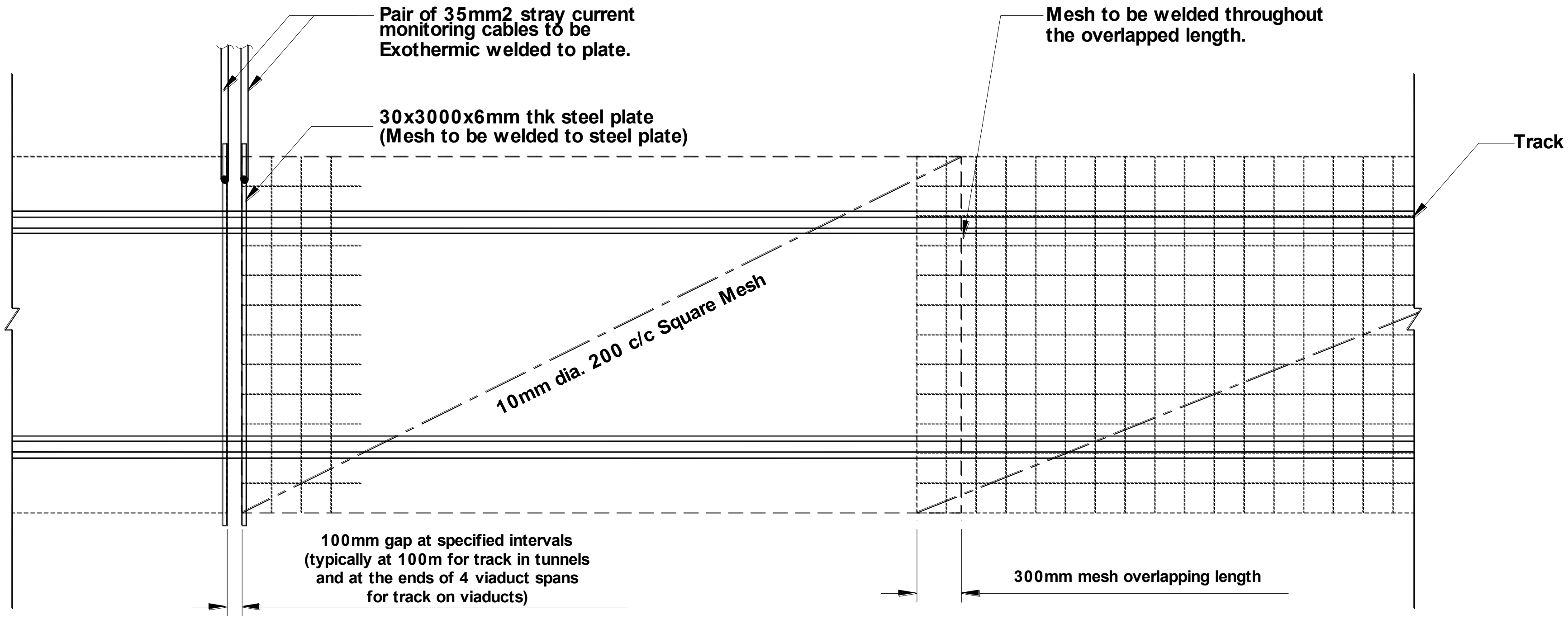


FIG. 14.8 – STRAY CURRENT COLLECTION MAT (PLAN VIEW)

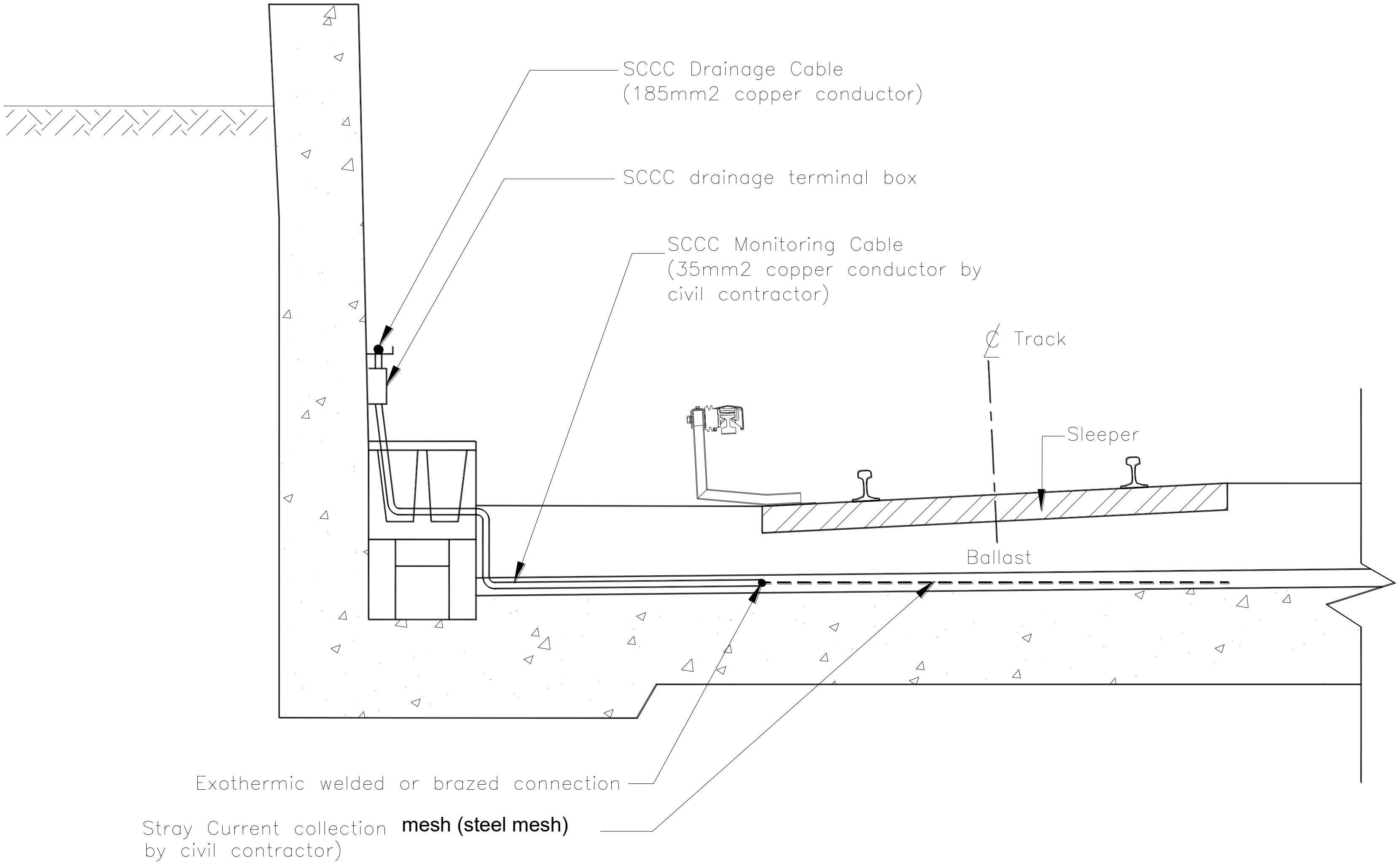


FIG. 14.9 – SECTION IN TRANSITION STRUCTURE

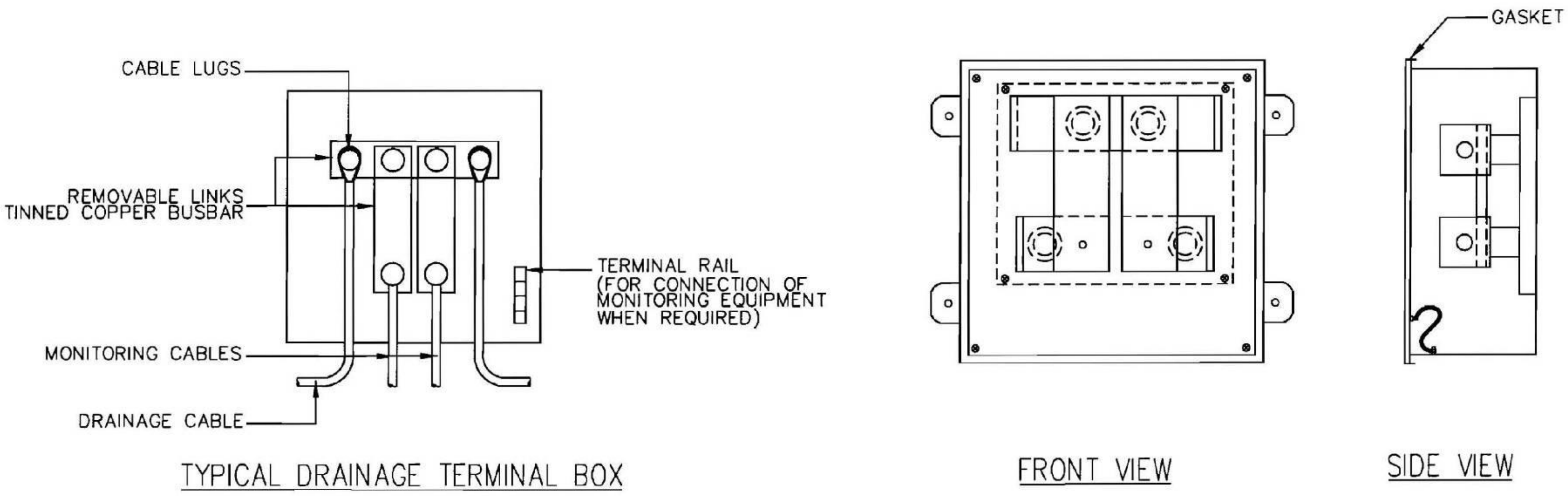


FIG. 14.10 – TYPICAL DRAINAGE TERMINAL BOX

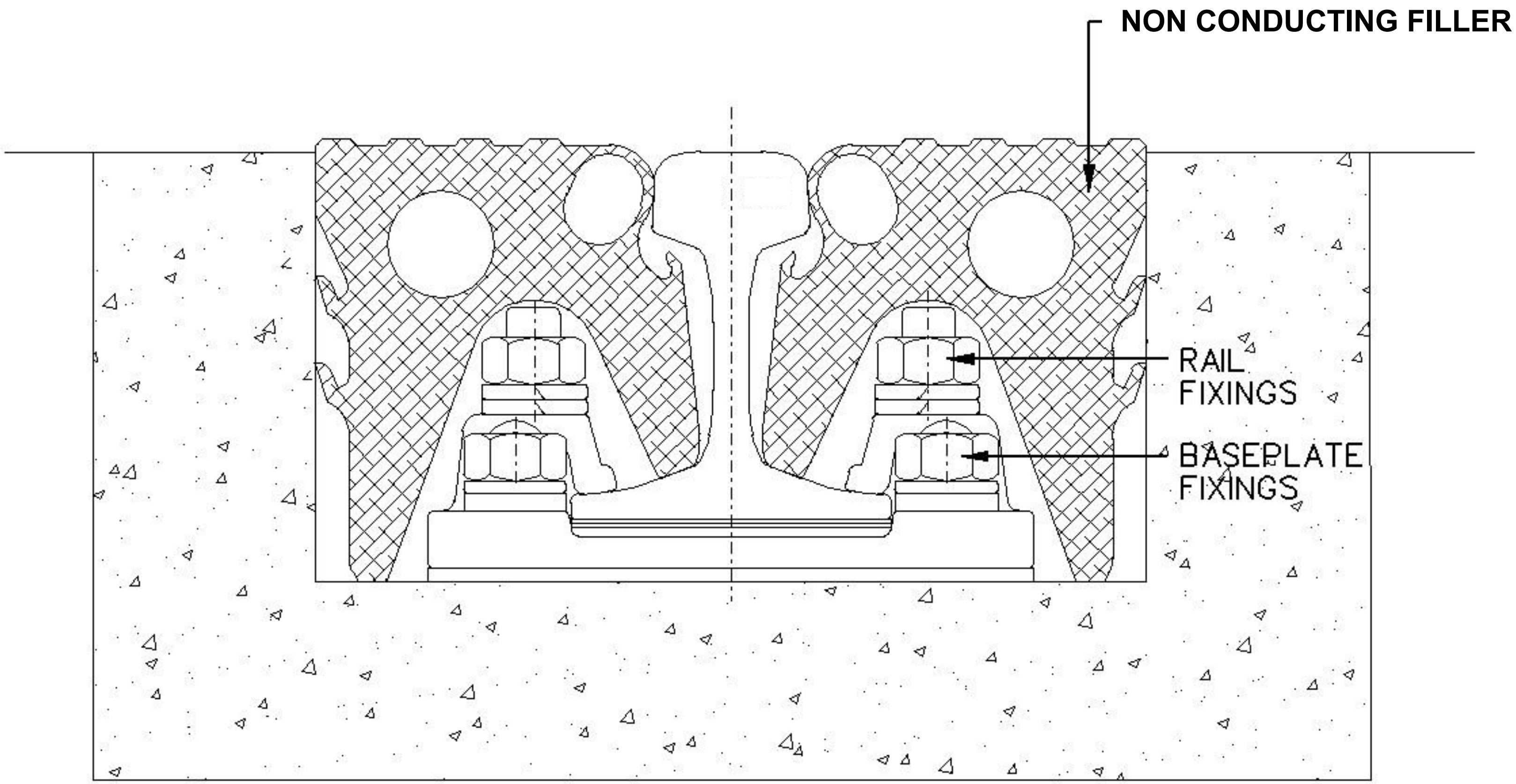


FIG. 14.11 CROSS SECTIONAL VIEW AT LEVEL CROSSING

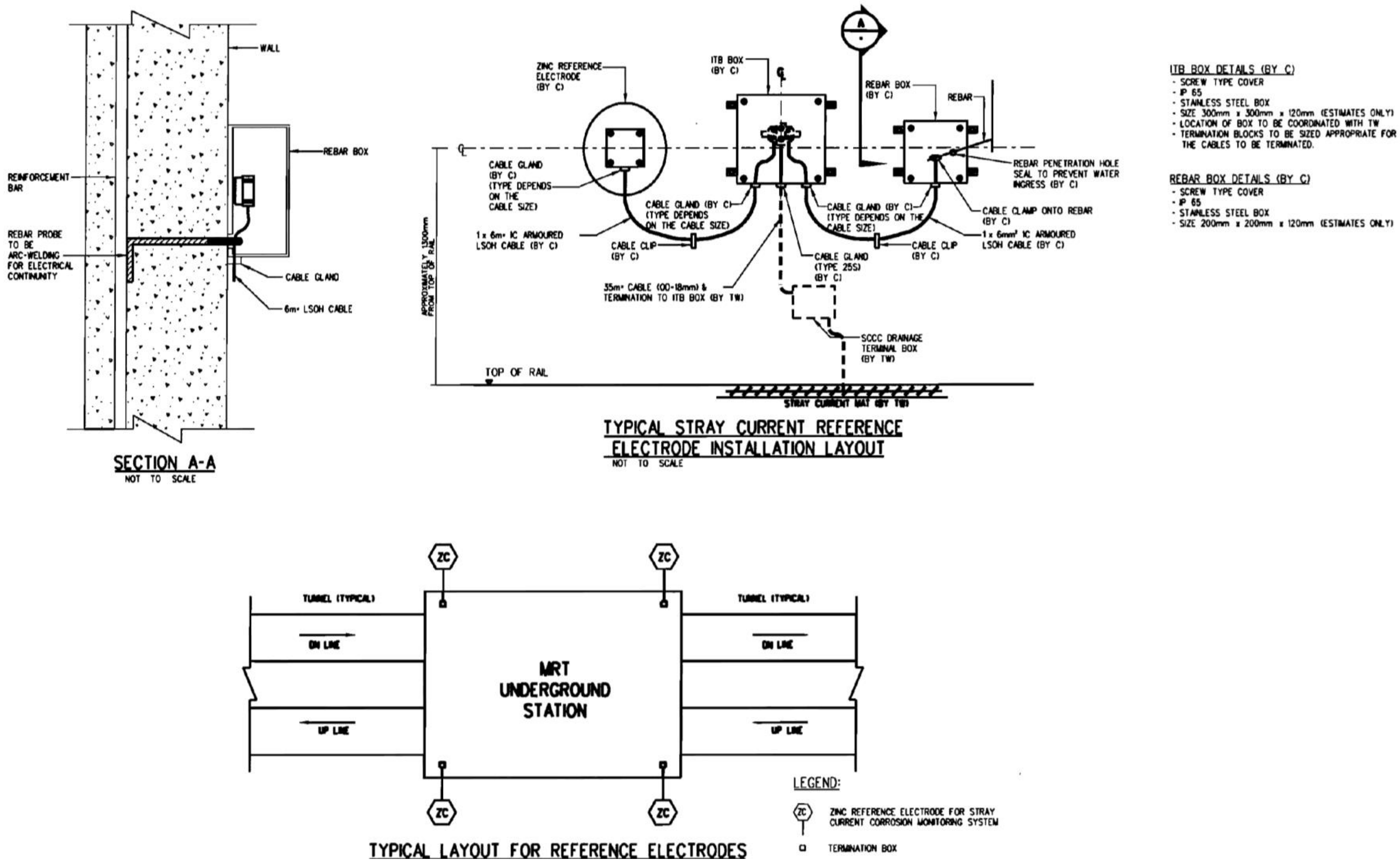


FIG. 14.12 – REBAR PROBE PROVISION DETAIL

CHAPTER 16

EMBEDDED RETAINING WALL FOR EXCAVATION

16.1 SCOPE

16.1.1 This Chapter covers the design requirements for Earth Retaining or Stabilising Structure (ERSS) and its support system used for excavation.

16.1.2 The ERSS in this chapter is specifically for embedded retaining wall for excavation.

16.2 STANDARDS AND CODES OF PRACTICE

16.2.1 The design of ERSS shall comply with the Building and Construction Authority's latest requirements.

16.2.2 The design of the ERSS shall also comply with SS EN 1997. Additionally, guidance from CIRIA Special Publication 95, CIRIA Reports C517 and C580 shall also be referred to in the design.

16.2.3 Reinforced concrete retaining walls including diaphragm walls, secant pile walls and contiguous bored piled walls shall be designed to SS EN 1992. Structural steel works including sheetpile walls, soldier pile walls and steel strutting system shall be designed to SS EN 1993.

16.3 ACTIONS

Surcharge loads as defined in Chapter 3 shall be used in the design of ERSS.

16.4 DESIGN REQUIREMENTS

16.4.1 General

The support system of the ERSS shall be robust to ensure lateral stability of the excavation and control ground and wall deformation.

16.4.2 Design & Analysis

16.4.2.1 The groundwater level shall be considered at ground level for each stage of construction unless otherwise accepted by the Engineer. Seepage analysis shall be considered at every stage of excavation to determine the change in piezometric pressure.

16.4.2.2 The design shall demonstrate that the toe-in depths of the ERSS provide adequate passive reaction. The wall shall neither be overstressed, nor deflect excessively. Adequate protection shall be provided against the ingress of ground water into the excavation.

16.4.2.3 The design shall include, where appropriate, adequate precautions against base heave in soft clays during construction. Failures such as base heave, inward yielding, piping or blow-out, etc. shall also be considered.

16.4.2.4 If the design of ERSS are analysed by finite element or finite difference methods, relevant empirical evidence from similar excavations shall be referred to in support of the conclusions of the analyses.

16.4.2.5 The excavation support system shall be designed for appropriate stresses obtained from analysis which consider the full construction sequence through to final removal of the temporary works. The analysis shall consider appropriate boundary conditions and progressive changes in pore water pressures.

16.4.2.6 The design shall allow for unplanned excavation at every stage of excavation in accordance with SS EN 1997.

16.4.3 Other Requirements

16.4.3.1 The design of ERSS shall take into consideration construction tolerance and deviations due to site constraints. The design assumptions, allowable tolerances and deviations shall be clearly stated in the design documents and working drawings.

16.4.3.2 Capping beams shall be provided for contiguous bored pile walls and secant pile walls. Capping beams shall be provided for diaphragm walls where the first strutting level is placed at a level more than 2m below the top of the wall.

16.4.3.3 Continuous walers shall be provided to the ERSS.

16.4.3.4 The use of recharge wells shall also be considered where excavation is carried out in soft clay so as to limit any damage to adjacent buildings, structures and utilities.

16.5 GEOTECHNICAL MODEL

16.5.1 The geotechnical model of the site shall be developed to provide a basis for the design of the works. The geotechnical model shall provide a coherent, three-dimensional (3-D) concept of how the major stratigraphic units and the groundwater relate to the structure being designed. The 3-D representation of the stratigraphy can be presented in contour plots or by other means acceptable to the Engineer. In hilly areas, the ground water level shall also be plotted using contours.

16.5.2 The geotechnical model shall be based on, inter alia: -

a) Information from a desk study;

b) Information obtained from site walkover studies;

c) Published information on the geology of Singapore and any available information on excavations or tunnels at or near the site

d) Foundation records of nearby buildings;

e) Site investigation carried out at the site and nearby, including boreholes, CPTs, insitu tests, geophysical surveys, trial pits and laboratory tests.

16.5.3 The design sections shall be derived from the geotechnical model, not based on individual borehole log. Allowance shall be made for the variability and uncertainty involved in developing the model.

16.5.4 The geotechnical model shall be updated during construction as and when additional information is available. Information that shall be considered shall include, inter alia: -

a) Additional site investigation results;

b) The logs from the installation of the wall, kingposts, piles, dewatering/pressure relief wells, monitoring instruments and any grouting or other geotechnical processes;

c) Mapping of exposed ground during excavation.

16.5.5 If there is a variance between the model on which the design is based and the updated model, the design shall be assessed and amended where necessary, to ensure that the construction can be carried out safely.

16.6 GEOTECHNICAL PARAMETERS

16.6.1 The geotechnical design parameters for each geological member or formation in each design section shall be based on: -

a) The results of laboratory testing;

b) The results of in-situ testing;

c) Inspection of the samples and cores;

d) Back-analyses of excavations of a similar scale in similar ground as appropriate and available.

16.6.2 The geotechnical design parameters used shall be clearly stated. Where the parameters have not been measured directly from field-testing, i.e., bulk and shear modulus, a clear explanation must be given to justify the values used.

16.6.3 The development of the parameters shall also consider published information on that particular member or formation. Site specific data shall take precedence over general published information. General published information can be used as a reference to assess the reliability of the site specific data, provide a framework for interpretation and to supplement the site specific data.

16.6.4 Any finite element program used for the analysis of the ERSS shall have the capability to demonstrate that the undrained shear strengths generated by the program are consistent with the strengths measured in the soil investigation (i.e. vane shear strengths and those derived from CPTs).

16.6.5 Where the strength and/or stiffness vary with depth, its profiles with depth must be determined. Any lateral variations (e.g. edge of reclamations) should also be assessed and considered.

16.6.6 For the strength and stiffness of each unit in an area, the characteristic values must be determined.

16.7 ULTIMATE LIMIT STATES (ULS)

16.7.1 General

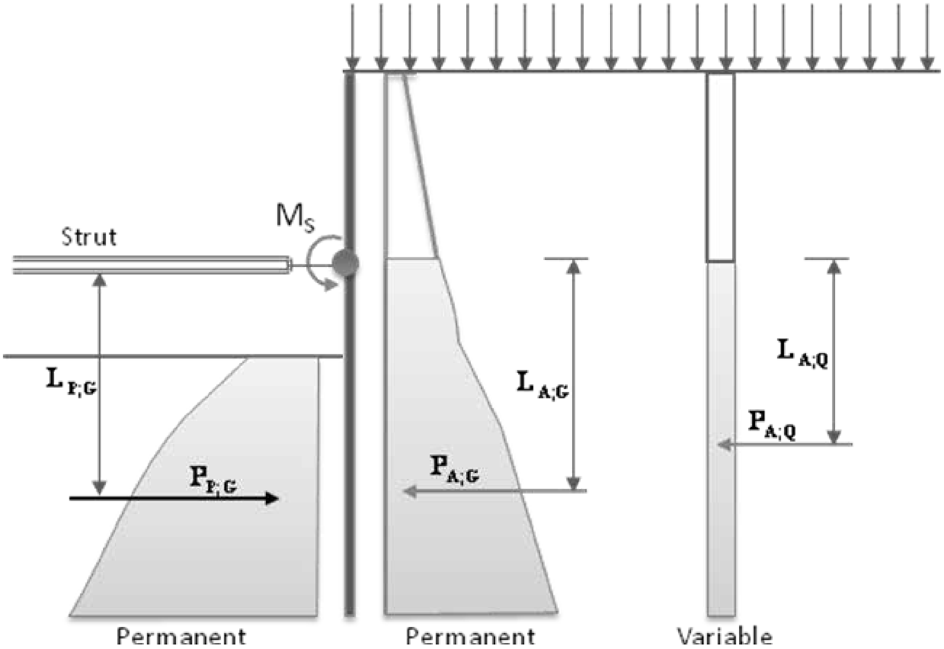
Checks of the ERSS shall be carried out to verify that the ultimate limit states as described in the SS EN 1997 are not exceeded, where the design value of resistance to an action (R_d) shall be greater than or equal to the design value of the effect of actions ($E_{d.}$), i.e. $R_d \geq E_d$.

16.7.2 Toe-in Depth

16.7.2.1 ULS verification check shall be carried out to determine the toe-in depth or embedment of the retaining walls.

16.7.2.2 The check shall be carried out based on the method shown in Figure 16.1.

Figure 16.1 – Check for Adequacy of Toe-in-depth



Limit State GEO: $R_d \geq E_d$

$$(P_{P;G} \times L_{P;G})/\gamma_{R;e} + M_s \geq \gamma_G \times P_{A;G}\, L_{A;G} + \gamma_Q \times P_{A;Q}\, L_{A;Q} \quad \text{Equation (1)}$$

Where,

$P_{A;G}$	is the total permanent force below the lowest strut at the active side
$P_{A;Q}$	is the total variable force below the lowest strut at the active side
$P_{P;G}$	is the total permanent force on the passive side

Characteristic values of the soil shear strength, c' and tanϕ' or c_u shall be divided by γ_M in deriving $P_{P;G}$, $P_{A;G}$ and $P_{A;Q}$;

$L_{A;Q}$	is the level arm for the total variable force
$L_{P;G}$ and $L_{A;G}$	are the lever arms for the total permanent passive force and active force, respectively
M_s	is the moment resistance of the wall at the lowest strutted level
γ_G and γ_Q	are the partial factors on permanent unfavourable actions and variable unfavourable actions, respectively
γ_M	is the partial factor for soil parameters
$\gamma_{R;e}$	is the partial factor for earth resistance

16.7.3 Base Stability

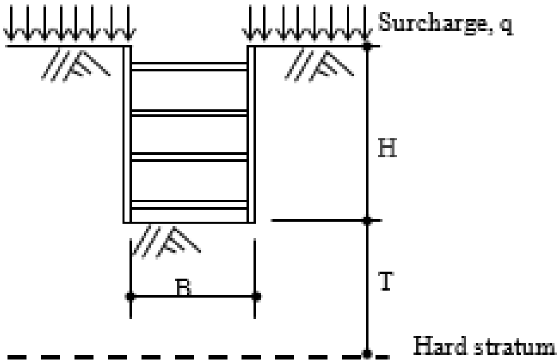
16.7.3.1 The design of ERSS for deep excavation shall include adequate measures against base heave.

16.7.3.2 ULS verification check shall be carried out to avoid the possible base failure in soft clay. This check shall be carried out based on the method in Figure 16.2(a): for narrow excavations ($B < H$), Figure 16.2(b): for wide excavations ($B \geq H$) with flexible walls and Figure 16.2(c): for wide excavations ($B \geq H$) with stiff walls in clay, where B is the width of excavation and H is the depth of excavation.

16.7.3.3 Checking of base stability shall be carried out not only for the final stage of excavation, but also for intermediate stages where layers of soft clay exist between ground level and the final formation level.

16.7.3.4 Where a stiff retaining wall is embedded into soil below the excavation depth, the Modified Terzaghi method shown in Figures 16.2(c) shall be used.

Figure 16.2(a) – Base Heave Checking for Narrow Excavations (B < H)



Limit State GEO: $R_d \geq E_d$

$$(N_c \times c_u)/(\gamma_{cu} \times \gamma_{R;v}) \geq \gamma_G \times \gamma\, H + \gamma_Q \times q \qquad \text{Equation (2)}$$

Where,

c_u is the characteristic value of undrained shear strength of the clay at the base

H is the depth of excavation

N_c is the bearing capacity factor

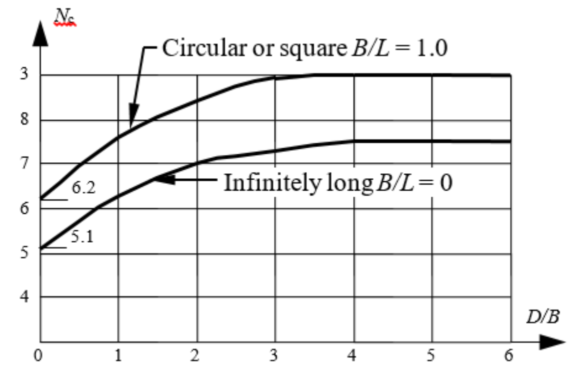
γ is the unit weight of the clay, q is the surcharge

γ_{cu} is the partial factor for c_u

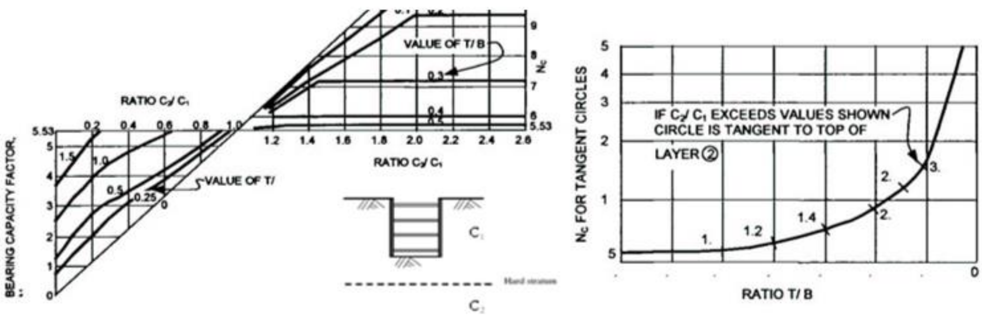
γ_G is the partial factor on permanent action

γ_Q is the partial factor on variable action

$\gamma_{R;v}$ is the partial factor for bearing resistance

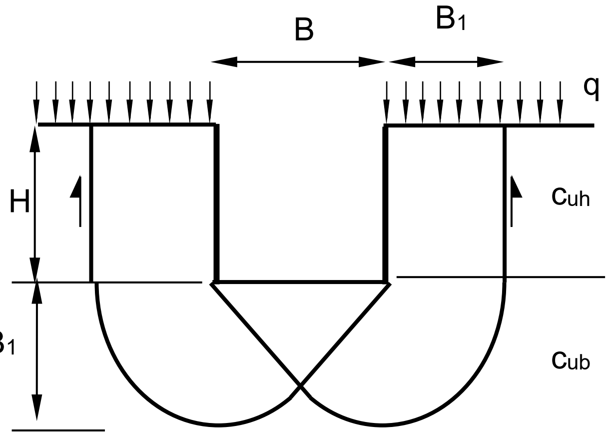


Bearing Capacity Factor for T>0.7B



Bearing Capacity Factor for T<or = 0.7B

Figure 16.2(b) – Modified Terzaghi Method for Wide Excavation with Flexible Walls, B ≥ H



Limit State GEO: $\mathrm{R_d} \geq \mathrm{E_d}$

$$(5.7\ \mathrm{c_{ub}}\ \mathrm{B_1} + \mathrm{c_{uh}}\ \mathrm{H})/(\gamma_{\mathrm{cu}} \times \gamma_{\mathrm{R;v}}) \geq \gamma_{\mathrm{G}} \times \gamma\ \mathrm{H}\ \mathrm{B_1} + \gamma_{\mathrm{Q}} \times \mathrm{q}\ \mathrm{B_1} \quad \text{Equation (3)}$$

Where,

B_1 is equal to 0.7B or depth to hard stratum, whichever is smaller

c_{ub} is the average shear strength (based on characteristic value) within the failure zone below the formation level

c_{uh} is the average shear strength (based on characteristic value) above the formation level

H is the maximum excavation depth

Q is the surcharge

γ is the average bulk unit weight above formation level

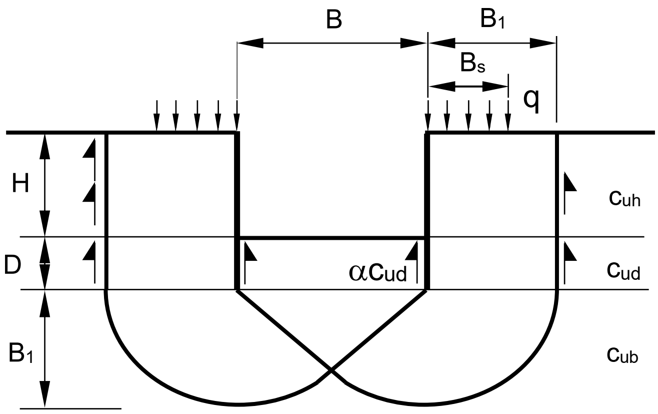
γ_{cu} is the partial factor for c_u

γ_G is the partial factor on permanent action

γ_Q is the partial factor on variable action

$\gamma_{R;v}$ is the partial factor for bearing resistance

Figure 16.2(c) – Modified Terzaghi Method for Wide Excavation with Stiff Walls in Clay B ≥ H



Limit State GEO: $\mathrm{R_d} \geq \mathrm{E_d}$

$$(5.7\ \mathrm{c_{ub}}\ \mathrm{B_1} + \mathrm{c_{uh}}\ \mathrm{H} + (1+\alpha)\ \mathrm{c_{ud}}\ \mathrm{D}) / \left(\gamma_{\mathrm{cu}} \times \gamma_{\mathrm{R;v}}\right) \geq \gamma_{\mathrm{G}} \times \gamma\ \mathrm{H}\ \mathrm{B_1} + \gamma_{\mathrm{Q}} \times \mathrm{q}\ \mathrm{B_s}$$

Equation (4)

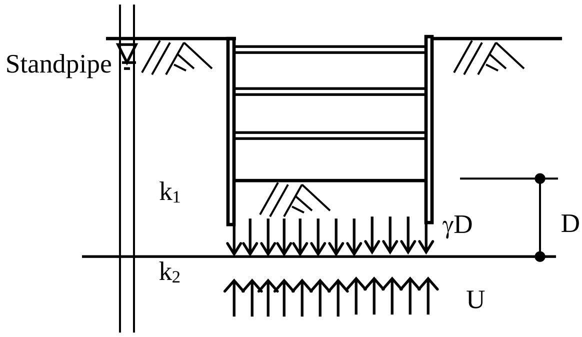
Where,

α	is the adhesion factor, α = 1
B_1	= 0.7B or (T – D) whichever is smaller
B_s	is the width of surcharge loading where $B_s < B_1$
C_{ub}, C_{uh} and C_{ud}	are the average shear strength (based on characteristic value) within the zones, respectively, as shown above
q	is the surcharge
T	is clay thickness below formation level
γ	is the average bulk unit weight above formation level
γ_{cu}	is the partial factor for c_u
γ_G	is the partial factor on permanent action
γ_Q	is the partial factor on variable action
$\gamma_{R;v}$	is the partial factor for bearing resistance

16.7.4 Hydraulic Uplift

16.7.4.1 ULS verification check shall be carried out to ensure that the base of the excavation would not fail by uplift. This check shall be carried out in accordance with Figure 16.3.

Figure 16.3 – Hydraulic Uplift checking



Limit State UPL: $R_d \geq E_d$

$\gamma_{G;stb} \times \gamma\, D \geq \gamma_{G;dst} \times U$ Equation (5)

Where,

D is the depth of soil cover from the formation level to the bottom of less permeable layer

U is the uplift force due to the excavation

γ is the saturated unit weight of the soil

$\gamma_{G;dst}$ is partial factor for a permanent destabilising action

$\gamma_{G;stb}$ is partial factor for a permanent stabilising action

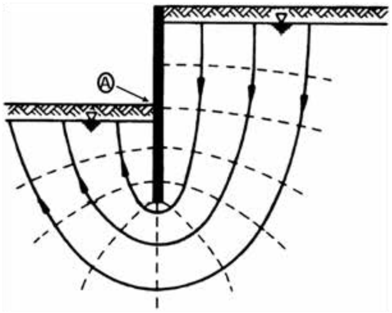
16.7.4.2 Where necessary and appropriate, Uplift Limit State must be verified for intermediate stages. Relief wells shall be provided inside the excavation where required.

16.7.4.3 Any development of pore pressures as the excavation progresses shall also be continuously monitored.

16.7.5 Hydraulic Failure by Piping

16.7.5.1 ULS verification check shall be carried out for hydraulic failure by piping, based on hydraulic gradient derived from seepage analysis and in accordance with Figure 16.4.

Figure 16.4 – Piping check



Limit State HYD: $R_d \geq E_d$

$$\gamma_{G;stb} \times i_{cr} \geq \gamma_{G;dst} \times i_{max(exit)} \qquad \text{Equation (6)}$$

Where,

i_{cr} is the critical hydraulic gradient

$i_{max(exit)}$ is the maximum exit gradient that will occur at point A

$\gamma_{G;dst}$ is partial factor for a permanent destabilising action

$\gamma_{G;stb}$ is partial factor for a permanent stabilising action

16.7.6 Stability of Slopes within Excavation

Analyses shall be carried out in accordance with SS EN 1997 to ensure that the internal slopes within the excavation area are stable and ensure that the excavation support system will not be affected.

16.7.7 ERSS Subjected to Vertical Actions

The ERSS subjected to vertical component of actions, including the weight of wall, capping beams, struts, slabs, walers and decking, the vertical component of actions from angled struts or anchors and any surcharges shall also be designed in accordance with the requirement in Chapter 6.

16.8 USE OF FINITE ELEMENT OR FINITE DIFFERENCE MODELLING TECHNIQUES

16.8.1 For each type of FE or FD model used, the analysis shall be checked using close-form solutions to verify the design approach and to validate the results.

16.8.2 All assumptions made in the FE or FD model shall be clearly stated. The limitations of the FE or FD programs shall also be highlighted.

16.8.3 It shall be demonstrated that the boundary conditions have no significant influence on the results of the program.

16.8.4 Evidence shall be provided to demonstrate a step-by-step approach to the modelling. The initial equilibrium conditions for a model shall be demonstrated. The rate of convergence and the final out-of-balance forces at each stage of the construction sequence shall also be provided.

16.8.5 Where structural elements have been used within a FE or FD model, their material properties, connection details and a justification for their use shall be provided.

16.9 SOIL MODELLING

16.9.1 General

16.9.1.1 The selection of an appropriate constitutive model and the failure criteria of the soil model used must be clearly identified, together with a justification for their use. The influence of the model type on the design shall be assessed.

16.9.1.2 For marine and fluvial clays that have a permeability of $1x10^{-8}$m/s or less undrained analysis using undrained shear strength shall be carried out. However, some degree of swelling within the excavation and consolidation outside the excavation will take place. These shall be assessed and its effects considered in the design.

16.9.1.3 For fluvial sands, drained analysis using effective stress parameters shall be carried out.

16.9.1.4 For other soil types, both drained and undrained analyses shall be carried out. The design shall be based on the more onerous case.

16.9.1.5 Wherever possible, relevant empirical evidence from similar excavations based on local Singapore experience must be referenced to, in support of the conclusions of the analyses.

16.9.2 Sensitivity Analysis

16.9.2.1 The design and any associated FE or FD modelling shall not rely merely on a “one-off” analysis, in which a single set of geotechnical parameters is used.

16.9.2.2 Sensitivity analyses shall be performed and submitted as part of the design to demonstrate that the design and the model are not unduly sensitive to variations in any of the input parameters. As a minimum, the following parameters shall be evaluated: -

a) Shear strength
b) Soil stiffness
c) Soil permeability
d) Pre-loading forces
e) Over-excavation
f) Consolidation parameters
g) Ground treatment strength and stiffness (where applicable)

16.9.2.3 Should the results be sensitive to any parameters, the selection of these parameters should be carefully evaluated, and the selected values for the parameters shall be conservative and be able to cover the likely scenarios occurring on site.

16.9.3 Back-Analysis

16.9.3.1 Back-analysis shall be done with careful reviews of all design assumptions and the monitored behaviour of the structures. The field monitoring data shall be reviewed comprehensively taking into consideration all the instruments in the vicinity.

16.9.3.2 The review of design assumptions shall include appropriateness of the models used in the original design, soil parameters, soil profiles encountered, construction sequences, drainage conditions on site and the wall stiffness. Any changes or revisions in the back-analysis shall not be arbitrary. There shall be clear rationale and substantiation, for example field or laboratory tests or additional evidence from construction records, for any changes in the model parameters.

16.9.4 Submission of Results

16.9.4.1 The results of the analyses shall be presented in a clear, concise format. An outline format shall be submitted to the Engineer for acceptance before any design submissions are made.

16.9.4.2 The submission shall include, amongst others, a clearly annotated printout of all legends used for the FE or FD model, soft copies of input files and plots showing the rate of convergence and final out of balance forces for each model.

16.10 STRUCTURAL DESIGN

16.10.1 The ERSS shall be designed to resist the bending moment and shear force predicted from Design Approach 1 – Combination 1 as minimum. As deemed necessary and appropriate, the designer shall design the ERSS against the actions from Design Approach 1 – Combination 2.

16.10.2 The ERSS shall be designed against the limit state of possible failure of a structural element such as a wall, anchorage, waler or strut or failure of the connection between such elements at each stage of the construction works, in accordance with SS EN 1997.

16.10.3 The design of the support system shall be checked for: -

a) Accidental actions not less than 50kN applied normal to the strut at any point in any direction, unless otherwise demonstrated by risk assessment;

b) One-strut failure;

c) Thermal actions of a minimum temperature difference of ±10°C.

16.10.4 The design of struts, walers and strut/waler connections shall take into account of eccentricity in transfer of load from the waler to the strut. The eccentricity shall be taken as 10% of the overall dimension of the strut in the vertical plan and not less than 30mm, regardless whether it is for single or twin walers.

16.10.5 All restraints to struts shall be designed to resist 2.5% of the axial force of the strut, in addition to any loads carried by the restraints. Any bracing system that provide restraint to more than one member shall be designed to resist the sum of the restraint forces from each member that they restrain, reduced by the factor α_m obtained from:

$$\alpha_{\mathrm{m}} = \sqrt{0.5\left(1 + \frac{1}{\mathrm{m}}\right)}$$

where m is the number of parallel members to be restrained.

16.10.6 Kingposts shall be designed as an unbraced column if no triangulated bracings are provided. The effective length of embedded kingpost shall be determined from analysis to derive the position of fixity below the ground.

16.10.7 Buckling and bearing checks shall be carried out for all strut/waler connections. Adequate stiffeners shall be provided to prevent web buckling. Where the flange through which the load is applied is not effectively restrained against lateral movement relative to the other flange, the effective length of the web shall be taken as greater than or equal to 1.5d, where d is the depth of web.

16.11 GROUND ANCHORAGES

16.11.1 ‘Ground anchorage’ shall be designed in accordance with SS EN 1997.

16.11.2 Any anchorage which has had an external load applied to it as part of the installation process shall be designed and constructed to be destressed before it is buried or covered by any part of the permanent works.

16.11.3 All anchorages, which are installed either wholly or in part outside the Railway Area or Road Reserve shall be made of non-metallic material or shall be designed to be removable as far as practicable to the acceptance of the Engineer.

16.11.4 The testing of ground anchorages as specified in the M&W Specifications shall be clearly indicated in the drawings.

16.12 GROUND TREATMENT

16.12.1 Where ground treatment is used, the design of the ERSS will be highly dependent on the strength and stiffness of the treated ground. Depending on the geometry of the excavation and the ground treatment, the highest moments or shear forces can result from the highest likely values of strength and stiffness for the treated ground. It is necessary to design against the full range of strength and stiffness values for treated ground.

16.12.2 The impact due to the scatter of strength and stiffness of treated ground shall be considered in the design. Sensitivity analysis shall be carried out taking into consideration the upper bound and lower bound strength and stiffness of the treated ground. The design shall base on the bending moment and force envelopes using the upper bound and lower bound parameters of treated ground.

16.12.3 There shall be no tension in the treated ground at all stages of construction.

16.12.4 The strain in treated ground shall be less than the strain at failure, unless the ‘post-peak’ behaviour is justified by testing and considered in the analysis.

16.12.5 The minimum requirements for the treated ground, such as strength and stiffness parameters, shall be clearly specified on the relevant drawings.

16.12.6 No gaps are allowed in retaining systems for excavations. Where gaps are unavoidable due to existing utilities, the following methods can be adopted depending on the utility dimensions, ground conditions and wall types.

Where diaphragm wall is chosen as the ERSS wall and if the utility is less than 2m depth and the utility gap of less than 2m, the diaphragm wall panels could be constructed from either side of the utility by shifting the utility cables. In other cases, the gap shall be grouted, if necessary by inclined drilling.

Jet grouting could be carried out for Kallang formation soils and other soft to stiff soils which do not have adequate strength to provide stability during excavation. Permeation grouting could be carried out in sandy soils to improve permeability to prevent water ingress and soil flow. In both cases, excavation shall be carried out in steps of 0.5m depth followed by shotcrete or lagging.

Where grouting could not be carried out, micropiles wall shall be carried out from below the utility.

The designer shall carry out detail design considering soil conditions and constraints at site.

16.13 Design FOR REMOVAL OF ERSS

16.13.1 All ERSS shall be designed for removal unless otherwise accepted by the Engineer. Where ERSS are required to be removed, they shall be designed such that suitable methods can be employed to minimise the ground movement resulting from extraction.

16.13.2 Temporary Works shall be designed such that there is no risk of damage to the Permanent Works during removal.

16.13.3 Unless otherwise accepted by the Engineer, all voids left in the ground due to the extraction of temporary works shall be immediately filled with grout. The grout mix and method of grouting shall be submitted to the Engineer for acceptance.

16.13.4 Where it is agreed that Temporary Works may be left in the ground they shall be designed so that there will be no risk of ground settlement or other deleterious effects as a consequence of decay of timber or other materials.

16.13.5 All ERSS that are not removed shall be designed to be broken out/ cut off to a depth of 2 metres below the finished ground level unless shown otherwise on the Authority’s Drawings. This shall also apply to all secant and diaphragm walls including guide walls.

16.13.6 The ERSS drawings shall indicate clearly the following:

a) Details of the construction sequence of the ERSS removal;
b) Identification of the ERSS that are not removed (if any); and
c) Provisions made in the design to satisfy the above requirements shall be detailed on the ERSS drawings.

16.13.7 The left-in ERSS shall also be shown on the as-built drawings. Any ERSS that are not removed shall be agreed by the land owner. The Contractor shall also obtain the landowners' written approval (for both State and non-State land) to leave any Temporary Works on their land.

References:

1. Bjerrum, L. & Eide, O. (1956). Stability of strutted excavations in Clay. Geotechnique, Vol. 6, No. 1, March 1956.

2. CIRIA (1993) Special Publication 95. The Design and Construction of Sheet-piled Cofferdams. Construction Industry Research and Information Association, London, UK.

3. CIRIA (1999) C517. Temporary Propping of Deep Excavations – Guidance on Design. Construction Industry Research and Information Association, London, UK.

4. CIRIA (2003) C580. Embedded Retaining Walls – Guidance for Economic Design. Construction Industry Research and Information Association, London, UK.

5. NAVFAC DM 7.2 (1982). Foundations and Earth Structures. Design Manual 7.2, Department of the Navy, Naval Facilities Engineering Command, Alexandria, VA 22332, USA.

6. Wong, K.S. & Goh, A.T.C. (2002). Basal heave stability for wide excavations. 3rd International Symposium on Geotechnical Aspects of Underground Construction in Soft Ground, Toulouse, France.

CHAPTER 18

IRRIGATION SYSTEMS

18.1 REGULATIONS, CODES AND STANDARDS

The design, manufacture, supply, installation, testing and commissioning of Irrigation Systems shall be governed by all applicable local codes of practice, regulations, standards and requirements issued by all local government and statutory authorities, agencies and service providers which shall include the following:-

(a)	National Parks Board	Nparks
(b)	Productivity and Standards Board	PSB

18.2 DESIGN CRITERIA

18.2.1 Manual irrigation system shall be designed to provide irrigation to the planting troughs, planting strips and landscape areas. The system shall comprise of coupling inlet, distribution pipe network and emitter systems e.g. sprinkler pipes, sprinkler heads and stream bubblers.

18.2.2 The mobile water tanker shall pump the water through the coupling inlet to deliver the water into the manual irrigation system. The pumping capability of the mobile water tanker shall govern the water flow rate and pressure head delivered into the system. Suitable pipe sizes and ball valve controls to distribute water to different respective irrigation zone at different time interval shall be incorporated in the system design.

18.2.3 Coupling inlet shall preferably be mounted to the column of the bridge and flyover, etc. Alternatively, it can be housed in a pit flushed to ground level with a hinged metal cover subject to the authority's acceptance. Location of the coupling point has to be within a radius of 8m from the mid-point of the lay-by and shall be easily and safely accessible by mobile water tanker.

18.2.4 Water filter shall be incorporated when sprinkler heads and bubblers are used. The filter shall be corrosion resistant with filter housing made of stainless steel grade 316. The filter shall be installed after the coupling inlet, easily accessed, removed and cleaned by flushing with water.

18.2.5 Pedestrian Overhead Bridge (POB)

The emitter systems shall be provided as follows:-

(a) POB less than 25m long, 25mm stainless steel sprinkler pipe (5mm diameter holes provided at 200mm centre to centre along the bottom of the pipe) fixed to the inner wall of the trough that abuts the platform and above the soil level.

(b) POB between 25m to 50m long, 25mm stainless steel sprinkler pipe (3mm diameter holes provided at 400mm centre to centre along the bottom of the pipe) fixed to the inner wall of the trough that abuts the platform and above the soil level.

(c) POB exceeding 50m long, multiple pipe system of 25mm stainless steel sprinkler pipe (3mm diameter holes provided at 400mm centre to centre along the bottom of the pipe) with ball valves to channel water to different pipes. The pipes shall be fixed to the inner wall of the trough that abuts the platform and above the soil level.

(d) One coupling point for each POB shall be provided unless otherwise accepted by the relevant authority.

18.2.6 Road Bridge, Vehicular Viaducts, Underpasses and Flyovers

The emitter systems shall be provided as follows:

(a) Planting troughs: Stream bubblers shall be used to irrigate the plantings. The bubblers are fed via distribution pipes buried within the soil and spaced evenly along the planting trough. The bubblers shall protrude out 50mm approximately above the final planting soil level.

(b) Planting strips and landscape areas: Sprinkler heads shall be used to irrigate the plantings. The sprinkler heads are fed via distribution pipes buried within the soil and located according to the planting arrangements.

(c) The irrigation coverage, operating capacity and installation location of the emitters shall meet the watering intensities and duration required for the plants subject to authority’s acceptance.

18.3 SPRINKLER HEADS AND STREAM BUBBLERS

(a) The operation of the sprinklers and the stream bubblers shall be vibration free and shall not produce any back splash.

(b) The sprinkler head and stream bubbler shall be constructed of UV-resistant plastic and stainless steel grade 304 or 316 parts.

(c) The sprinkler head shall produce a fine spray of water of even distribution over the entire area of coverage. Low precipitation irrigation sprinklers are preferred. Each pop-up body shall have a filter screen and a wiper seal.

(d) The bubblers shall provide consistent and precise flow rate compensating for pressure variation caused by terrain elevation or friction loss.

18.4 PIPES AND FITTINGS

18.4.1 Materials

(a) The main and lateral underground pipes shall be heavy duty UPVC conforming to SS 141, Class E.

(b) Stainless steel pipes and fittings conforming to BS4127: Part 2, Grade 304 shall be used for the exposed piping.

(c) Heavy duty UPVC conforming to SS 141, Class E pipe sleeves shall be provided where pipes pass through walls or run under road (concrete encasement of 100mm thickness shall be provided). There shall be a minimum of 20mm clearance between pipe outside diameter and sleeve inside diameter. The space caulked with a soft non-setting waterproof mastic compound to give airtight seal.

18.4.2 Pipe Supports

(a) All steel hangers, supports, anchors, bolts, nuts, washers shall be hot dip galvanised.

(b) Supports shall be provided at spacings not exceeding the values specified in the table below unless otherwise indicated:-

MATERIAL OF PIPE	**DIAMETER OF PIPE IN MM**	**MAX. SPACING OF SUPPORTS** (Hori. run in m or vert. run in m)
Stainless Steel Pipe	20 - 25	2.5
	32 - 50	3.5

18.5 PIPE INSTALLATION

(a) All pipes shall be rigidly anchored and supported with hangers and supports. Each length of pipe and each fitting shall be thoroughly cleaned out before installation.

(b) Pipework shall be provided with sufficient flexible coupling for all forces and movements and detachable couplings for easy dismantling and assembly during maintenance.

(c) Any open ended pipe or pipe connection left overnight or for any considerable period shall be protected from the entry of dirt, sand, etc. by the fixing of approved plastic or galvanised iron sheet cap securely held.

(d) All exposed pipework shall have clearance of approximately 75mm from walls, ceilings or slabs. Pipe joints or fittings shall not be permitted within the thickness of walls and slabs.

(e) Pipework shall be installed with correct falls gradient to ensure adequate draining.

(f) A gasket, conforming to ANSI B16.21 and ASTM D2000, shall be installed between clamp and pipe or dissimilar materials.

(g) All pipe works shall be equipotential bonded to the earthing system as per requirement under the CP 5, Electrical Installations.

(h) Puddle flanges cast in wall shall be used when water-tightness is required.

(i) Exposed pipes shall be routed and installed to blend into the structures aesthetically. Pipes shall not be cast into any structural elements/ members.

CHAPTER 19

INSTRUMENTATION

19.1 SCOPE

19.1.1 This Chapter covers the Authority's requirements for the design and selection of instrumentation and monitoring works.

19.1.2 The Building and Construction Authority (BCA) and other regulatory bodies also require the installation and monitoring of instruments for excavations, tunnelling, piling and other construction works. Compliance with this Chapter may not be sufficient to fully satisfy these regulatory bodies. Both the instrumentation required under this section and any additional instrumentation required to satisfy the regulatory bodies shall be designed for and shown on the drawings.

19.2 INSTRUMENTATION REQUIREMENTS

19.2.1 The instrumentation shall be designed and selected to: -

a) Verify the assumptions made in the design.

b) Provide confirmation of the predicted behaviour of the support system during excavation or tunnelling and other construction works.

c) Enable the assessment of the effects of the works on buildings, utilities and other structures.

d) Provide a record of performance.

e) Enable construction to be carried out safely at every stage.

f) Enable appropriate contingency measures to be implemented in time where required.

19.2.2 In order to meet these requirements, the instrumentation design shall include the notional minimum monitoring specified herein and any such additional instrumentation as is necessary for safe construction works.

19.3 INSTRUMENTATION PLANS

19.3.1 Instrumentation plans shall be prepared clearly indicating the location and installation height/depth (SHD) of all instrumentation. Sectional drawings shall show the extent and depth of permanent and temporary works and the strata where the instruments are to be installed. All instruments shall be clearly identified in a unique alpha-numeric sequence using the instrumentation nomenclature as defined in the LTA's relevant database, such as Tunnelling and Excavation Monitoring System (TEMS). Each drawing shall show all of the instrumentation to be installed within the area covered in the drawing. The instruments within the influence zone beyond the contract boundary shall also be shown.

19.3.2 Where multi-sensor instruments are used, they shall be clearly shown in the drawings with the height/depth (SHD) and direction of each individual sensor.

19.3.3 Where buildings, structures or utilities requiring protection works, a detailed drawing indicating the structural and utilities layout, and the exact location of all monitoring points, including the height/depth (SHD) of the proposed instrument shall be prepared for each individual building, structure or utility.

19.3.4 In addition, sections shall also be included in the drawings indicating clearly the extent of any existing features, cracks etc., requiring monitoring.

19.3.5 Instrumentation tables in the drawings shall include the following information for each instrument: -

a) Instrument type

b) Unique instrument number

c) Sensor number

d) Proposed location (i.e. coordinates and reduced level of all sensors)

e) Proposed reading frequency during key stages of the works identified in the design

f) Review levels for all sensors (see Clause 19.7)

19.4 MONITORING ARRAYS

Monitoring arrays shall extend along the full length of the works, including an additional length of at least 50m beyond contract boundaries.

19.4.1 Monitoring for Excavations

19.4.1.1 For excavations deeper than 4.0 m, the following notional monitoring arrays using Type A, B and C as shown in Figures 19.1 and 19.2 shall be provided: -

a) One Type A array for every 25m (and part thereof) of perimeter wall or slope;

b) One Type B array for every 100m (and part thereof) of perimeter wall or slope; and

c) One Type C array for every 500m (and part thereof) of perimeter wall or slope.

19.4.1.2 Taking into account the nature and sequence of construction, Types B and C arrays shall be located in areas of particular concern, such as near sensitive buildings, structures or utilities, or in difficult ground condition. Types B and C arrays are not required to be evenly spaced.

19.4.1.3 Regardless of whether the excavation is being carried out in multiple stages or areas adjacent to each other, the distances used to assess the number of monitoring arrays shall be taken as the plan distance around the perimeter of the whole excavation area measured at the top of the slope or retaining wall.

19.4.2 Monitoring for Tunnels

19.4.2.1 For tunnelling (including pipe jacking), the following notional monitoring arrays using Type D, E and F, as shown in Figures 19.5 and 19.6, shall be provided: -

a) One Type D array for every 25m (and part thereof) of single tunnel;

b) One of Type E array for every 200m (and part thereof) of single tunnel; and

c) One of Type F array for every 1000m (and part thereof) of single tunnel, where the tunnels have an excavated diameter in excess of 3m.

19.4.2.2 Taking into account the nature and sequence of construction, Types E and F arrays shall be located in areas of particular concern, such as near sensitive buildings, structures or utilities, or in difficult ground condition. In addition, Type F array shall be located before entering the vicinity of the critical structures. The Contractor shall review the monitoring results from all instruments in relation to his tunnelling Key Performance Indicators and submit the report to the Engineer. Types E and F arrays are not required to be evenly spaced.

19.4.2.3 The distance shall be measured along the centreline of the tunnels and shall be based on the total length of tunnelling required under the contract.

19.4.2.4 For tunnels with excavated diameter equal to or less than 3m, the number of Type E monitoring arrays can be reduced by one half of that defined above, and Type F arrays are not required.

19.4.2.5 For twin or multiple tunnels where the monitoring arrays overlap, the instruments over the overlapping arrays can be reduced to provide the same level of monitoring for each of the tunnels as for the single tunnel.

19.4.2.6 Subsurface instruments shall not be installed around the tunnel within one tunnel diameter or 6m whichever is smaller.

19.4.3 Monitoring for Areas of Ground Treatment

A grid of settlement points to monitor heave or settlement shall be established over any area of ground treatment extending to at least 10m beyond the edges of the ground treatment area.

19.4.4 Monitoring for Vibration

Vibration monitoring shall be carried out on affected structures identified by the designer where the construction activity is likely to result in vibrations, for example, piling, blasting, tunnelling etc.

19.4.5 Monitoring of Struts and Ground Anchors

19.4.5.1 At least 15% of all struts and ground anchors shall be monitored for load using strain gauges and/or load cells. At least 25% of the struts and anchors monitored using strain gauges shall also be monitored using load cells. Where strain gauges are used, a minimum of 2 strain gauges coupled with temperature monitoring shall be installed at each monitoring location for struts, as shown in Figure 19.7.

19.4.5.2 The strut and anchor monitoring locations shall be selected so that: -

a) every strut/anchor level at the selected locations is monitored; and

b) the strut/anchor monitoring location coincides with a Type B or C monitoring array.

19.4.5.3 Where additional struts are installed during construction, these struts shall also be monitored.

19.4.6 Monitoring of Permanent Works

All structures founded on raft foundations shall be monitored for settlement after casting of the raft foundation until the completion of the Works.

19.4.7 Monitoring of Buildings and Structures

19.4.7.1 The relevant points of the buildings and structures, where any part of the building or structure fall within the monitoring zones defined in Figure 19.3, shall be identified by the designer and monitored for distortion of the building or structures.

19.4.7.2 Tensile strains shall be monitored by extensometers or by optical survey for all buildings with predicted tensile strain greater than 0.1% and for any building subjected to protection works as required by the impact assessment.

19.4.7.3 Crack monitoring shall be carried out for existing cracks, new cracks occurring during the works, and movement joints.

19.4.7.4 For buildings on mixed foundations, settlement points shall be placed on both sides of the interface between different foundation types.

19.4.8 Monitoring of Utilities

All utilities within the monitoring zone defined in Figure 19.4 shall be monitored with settlement points placed on or just above the utility, at intervals of not more than 25m. Utilities that are less than 4m below ground level should be monitored with points on the utility or as agreed with the relevant utility agency, while utilities more than 4m below ground can be monitored with points placed 2m above the utility or as agreed with the relevant utility agency. Joints in electrical cables with voltage higher than 22kV and in joints fibre optic cables shall be monitored for movement across the joint.

19.4.9 Monitoring for Tunnelling Under Buildings or Structures which are in Use

Where any part of the tunnel is directly or partially underneath any part of a building or structure that is in use, the following shall be implemented: -

a) A comprehensively monitored zone shall be set up within the area 50m before the tunnel reaches the building. The monitoring is required to confirm that the tunnelling is being carried out in such a way that the settlements are less than or equal to that expected from the design. The zone shall include at least two type E arrays, at approximately 25m and 50m respectively from the building to confirm the expected volume loss during tunnelling.

b) The building shall be monitored in accordance with Cl. 19.4.10.

c) The instruments in the zone and on the building shall be read when they are within 25m (in plan) of the location of the face of the tunnel. The monitoring data shall be assessed at least once per ring of advance or once per three hours, whichever is more frequent.

d) There shall be a direct communication channel from the tunnel to the Engineer assessing the data of the monitoring.

19.4.10 Monitoring in Real Time

19.4.10.1 Monitoring in real time shall be provided for the following: -

a) Structures on mixed foundations which may be affected by the works and where a part of the structure is on foundations underlain by soft or loose soils.

b) Structures undergoing underpinning.

c) Structures subject to compensation grouting.

d) Structures subject to protection works.

e) Buildings and structures which are in use and are located directly over a bored tunnel drive.

f) All struts and ground anchors monitored for load.

g) Vibration monitoring where any construction activity is carried out near to sensitive buildings, structures, utilities or equipment.

h) Any other structure where real time monitoring is considered necessary by the Engineer.

i) Operating rail tracks which are in the vicinity of the construction works.

19.4.10.2 Real time monitoring system for buildings or structures shall include one or more of the following systems: -

a) Either Electrolevels beams or Micro-Electro-Mechanical-System (MEMS) sensors with a backup system of precise levelling pins at both end of each beam

b) Automated total station(s) reading the movement of survey prisms.

19.4.10.3 Depending on the nature of the structure and the critical activity, Linear Variable Differential Transformers (LVDTs), tiltmeters, automatic inclinometers, piezometers, extensometers and / or other types of instruments may be used to provide additional real time information.

19.4.10.4 Instruments monitored in real time shall be connected to data logging equipment and the data shall be continuously accessible on computers in the Engineer’s site office, QP(S) site office and the main contractor’s site office.

19.4.10.5 The monitoring data from the real time monitoring shall be processed and displayed automatically, such that the displayed data represents the most recently taken readings.

19.4.11 Additional Monitoring Requirements of Sprayed Concrete Lining for Tunnels

19.4.11.1 General

The monitoring of the Sprayed Concrete Lining (SCL) works shall provide a database to verify the SCL design based on the actual ground mass behaviour.

19.4.11.2 Deformation Monitoring of the SCL

19.4.11.2.1 Deformations of the primary lining shall be monitored by observing movement of measuring bolts or prisms installed immediately after the excavation. Measuring bolts and prisms shall be fixed during the course of tunnel advance, after the application of the shotcrete within 1 m of the tunnel face. The following measurements shall be taken: -

a) Optical survey of prisms using high precision total station; and

b) Distance between measuring bolts using tape extensometer.

19.4.11.2.2 Deformation measurements shall be taken in arrays at intervals, subjected to the acceptance of the Engineer, along the tunnel as well as at junctions, bifurcations and in adjacent openings. The position of the measuring bolts and prisms shall be adjusted to suit the excavation sequence.

19.4.11.3 Stress Monitoring of the SCL

Strain gauges shall be installed in circumferential direction in order to assess the load build-up and distribution on the primary lining. The strain gauges shall be located at mid depth of the shotcrete as far as practicable.

19.5 READING FREQUENCY FOR MONITORING INSTRUMENTS

19.5.1 The design shall include tables giving reading frequencies for all instruments. The frequency of reading the instruments may be varied, depending on the type of instrument and the relationship between the instrument and areas of current activity. Criteria for increasing or decreasing the frequency of reading and for ceasing monitoring shall also be given, subjected to the acceptance of the Engineer.

19.5.2 For real time monitoring such as during active tunnelling under buildings or critical structures, underpinning and other critical works, the designer shall assess the appropriate interval between successive readings on critical sensors or points.

For the monitoring to be considered ‘real time’, the time between successive reading of the same sensor or point must be less than 4 hours.

19.6 ACCURACY AND RANGE OF MONITORING INSTRUMENTS

19.6.1 The accuracy and reading range of all of the monitoring instruments shall be specified as part of the design.

19.6.2 For instruments monitored in real time using automated total station monitoring, provisions shall be made for regular manual monitoring to confirm the continued accuracy of the system.

19.6.3 The depth of inclinometers shall extend beyond the influence zone of the tunnels and excavations being monitored to the point of fixity. As a minimum, inclinometers shall be taken down to at least: -

a) 2m into hard stratum; or

b) 3m below the toe level of the retaining wall (for excavations) or 3m below invert level (for tunnels) whichever is deeper.

19.6.4 Where Vibrating Wire Piezometers are specified in Figures 19.1, 19.2, 19.5 and 19.6, Casagrande Piezometers may be used as an alternative where they can be demonstrated to be sufficiently responsive to changes in piezometric pressure in the surrounding ground to provide representative readings.

19.6.5 Ground settlement points shall be installed so that the movement of the ground can be measured. Where settlement points have to be installed through road slabs, pavements or other surface structures, they shall be anchored into the ground with sleeves provided so that the measured settlement is not affected by the presence of the structure.

19.6.6 The design shall include protection for all instruments, to ensure that they are suitably protected against accidental damage, vandalism, and adverse climatic conditions, as stated in the M&W Specification.

19.7 REVIEW LEVELS

19.7.1 Prior to the start of construction, review levels shall be assigned to every instrument installed or to be installed. The values for review levels shall form part of the design.

19.7.2 There shall be two review levels: Alert Level and Work Suspension Level.

19.7.3 The Alert level is a reading at a predetermined level prior to the Work Suspension level or required for compliance with the Contract.

19.7.4 The Work Suspension level is defined as the highest (or the lowest as appropriate) reading anticipated based on the design at serviceability limit state.

19.7.5 In addition, levels predicted in the design shall also be set for each stage of construction. If any predicted level is exceeded, the design shall be reviewed with new analyses to be undertaken to estimate the final performance, and to re-assess and evaluate the design adequacy for all remaining stages.

19.7.6 Where appropriate, review levels shall be set for both positive and negative readings, e.g. for ground movement points, two review levels shall be set for settlement and two for heave.

19.7.7 If the Alert level is reached, the related instrumentation data and the ERSS design shall be reviewed. If the review concludes that the ERSS performs as per design, work can proceed. If the review shows that the Work Suspension level is likely to be exceeded, the design shall be re-assessed and additional measures shall be implemented to ensure that the Work Suspension level will not be exceeded.

19.7.8 If any reading exceeds the Work Suspension level, the related part of the Works shall be made safe and ceased until a full re-assessment of the original design is completed and remedial measures are implemented to ensure the safety of the works.

19.7.9 The full re-assessment shall be carried out comprehensively with careful reviews of all design assumptions and the actual monitored behaviour of the structures. All monitored parameters shall be checked when back-analysis is carried out during the full re-assessment, and the back analysis shall not be limited to curve fitting of wall deflections. There shall be clear evidence of improvement between computed and measured behaviour to date, i.e. from the beginning up to the stage where back analysis is carried out.

19.7.10 The review of design assumptions during back analyses shall include appropriateness of the models used in the original design, soil parameters, soil profiles encountered, construction sequences, drainage conditions on site and the ERSS stiffness. Any changes or revisions in the back analysis shall not be arbitrary. There shall be a clear rationale and substantiation, for example field or laboratory tests or additional evidence from construction records, for all changes in the model parameters.

19.7.11 Changes may be proposed to the values selected as review levels after a full re-assessment. Changes to review levels shall be justified on the basis of observed performance and the full re-assessment, and shall be submitted for the acceptance of the Engineer.

List of Figures:

Figure 19.1 – Monitoring Arrays for Supported Excavations

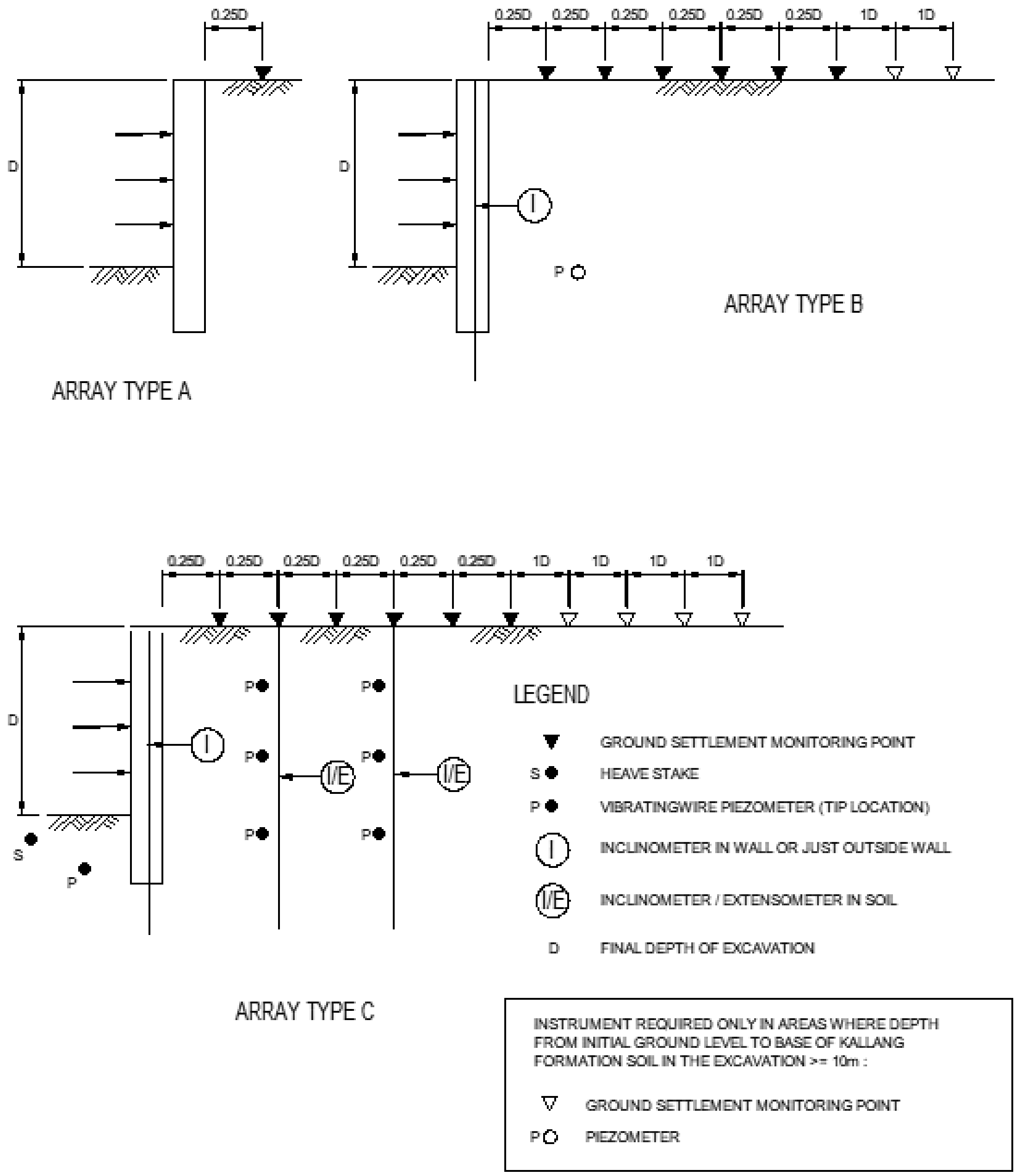
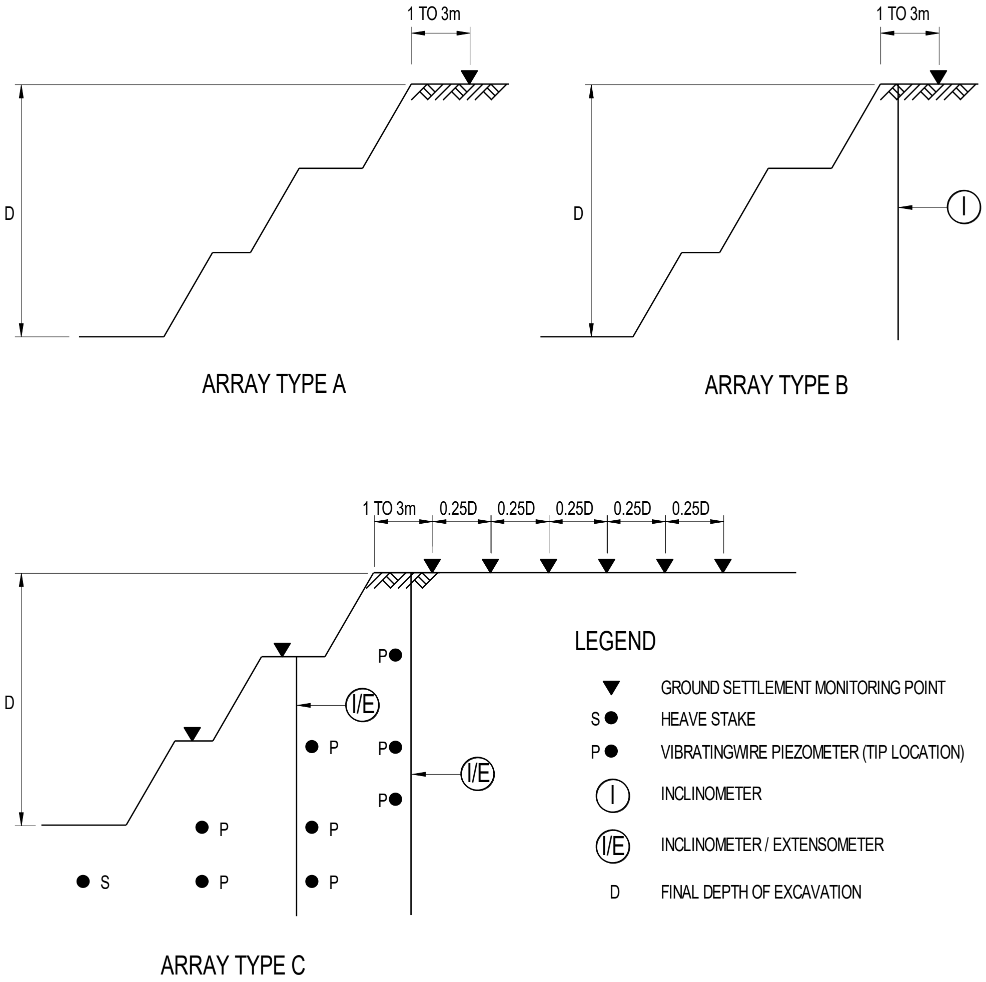


Figure 19.2 – Monitoring Arrays for Unsupported (Open) Excavations

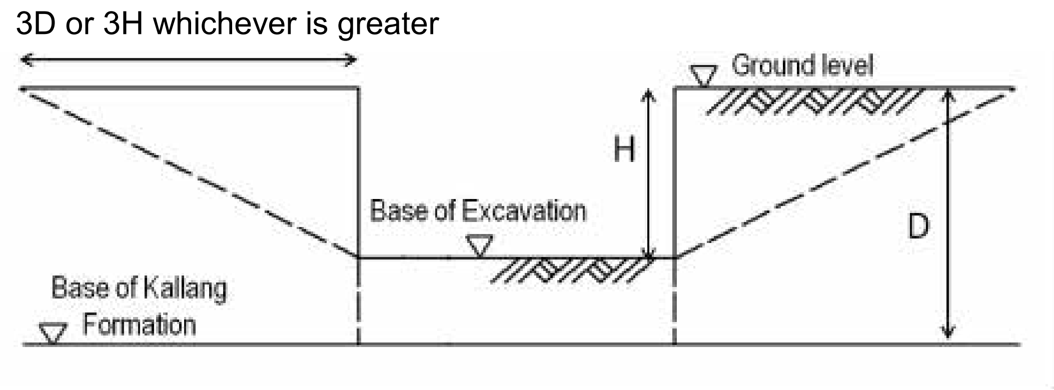


NOTE:

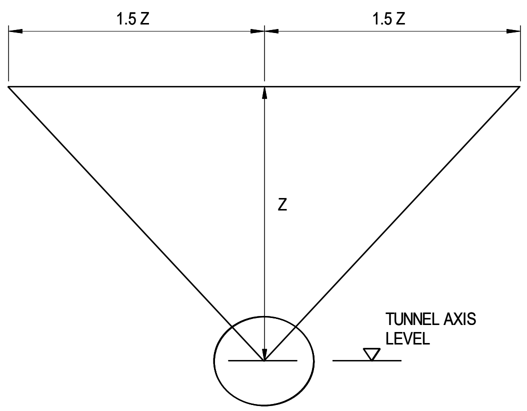
BENCHES ARE ONLY SCHEMATIC. MINIMUM INSTRUMENTATION NUMBERS ARE INDEPENDENT OF NUMBER OF BENCHES EXCEPT THAT ONE SETTLEMENT POINT IS TO BE PLACED ON EACH BENCH FOR TYPE C ARRAYS.

Figure 19.3 – Monitoring Zones for Building and Structures

1. EXCAVATIONS



2. TUNNELS

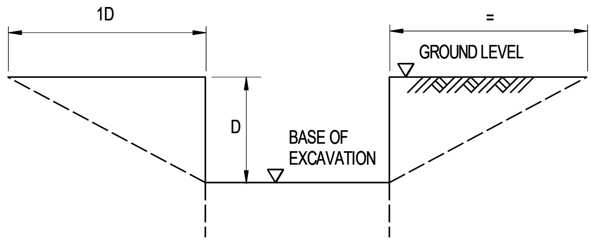


Note: For tunnel depth deeper than 30m, narrower zone can be adopted subject to the acceptance of the Engineer.

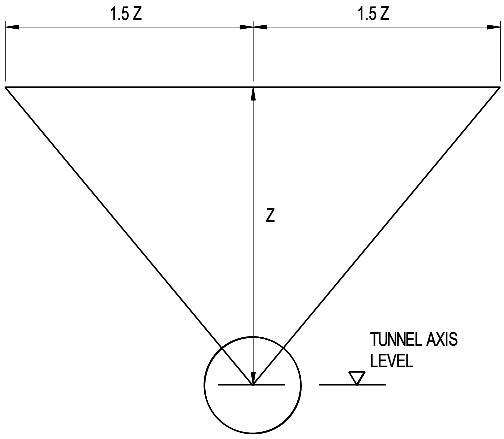
The monitoring zone shown above shall be reviewed by the designer and adjusted accordingly based on his assessment on the influence zone of the works

Figure 19.4 – Monitoring Zone for Utilities

1. EXCAVATIONS

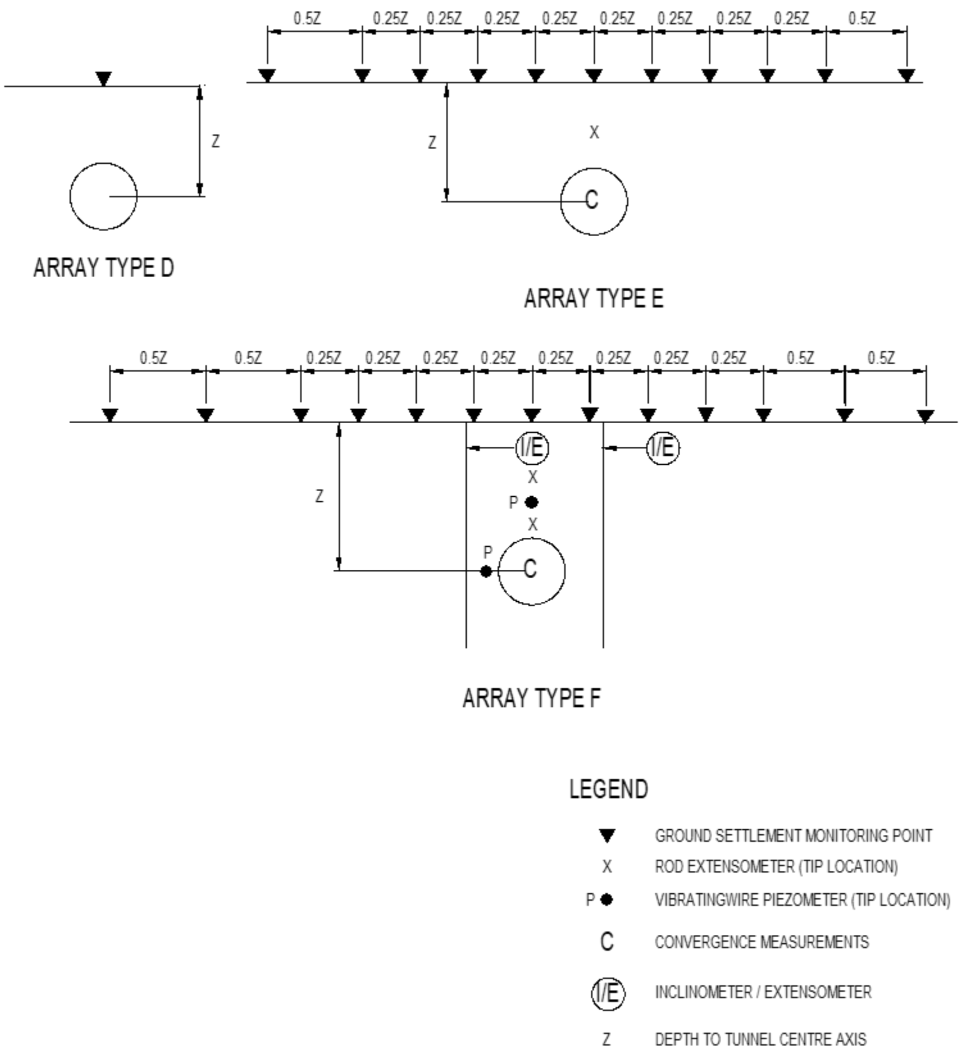


2. TUNNELS



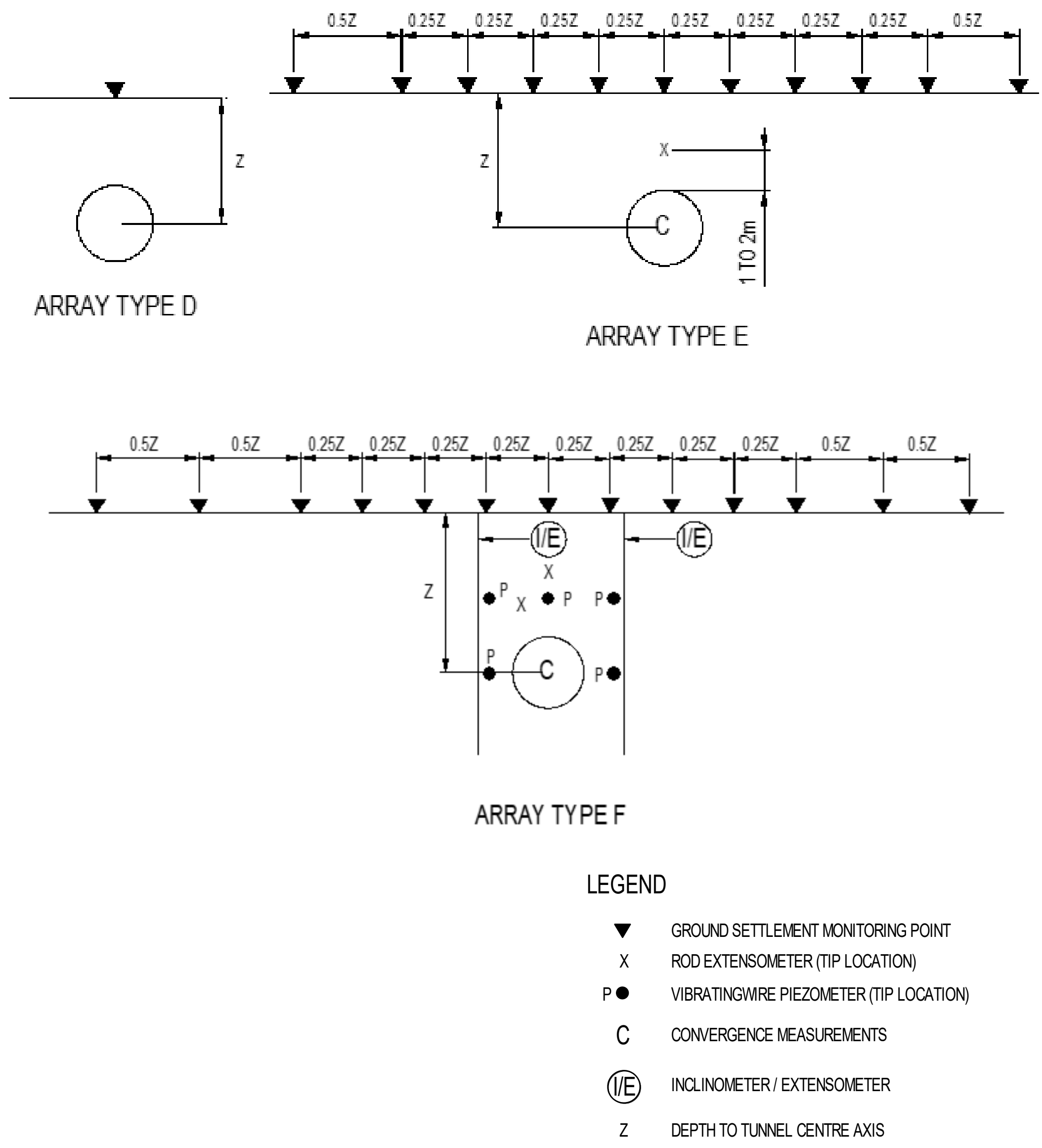
The monitoring zone shown above shall be reviewed by the designer and adjusted accordingly based on his assessment on the influence zone of the works

Figure 19.5 – Monitoring Arrays for Tunnels not in Kallang or Tekong Formation Soil



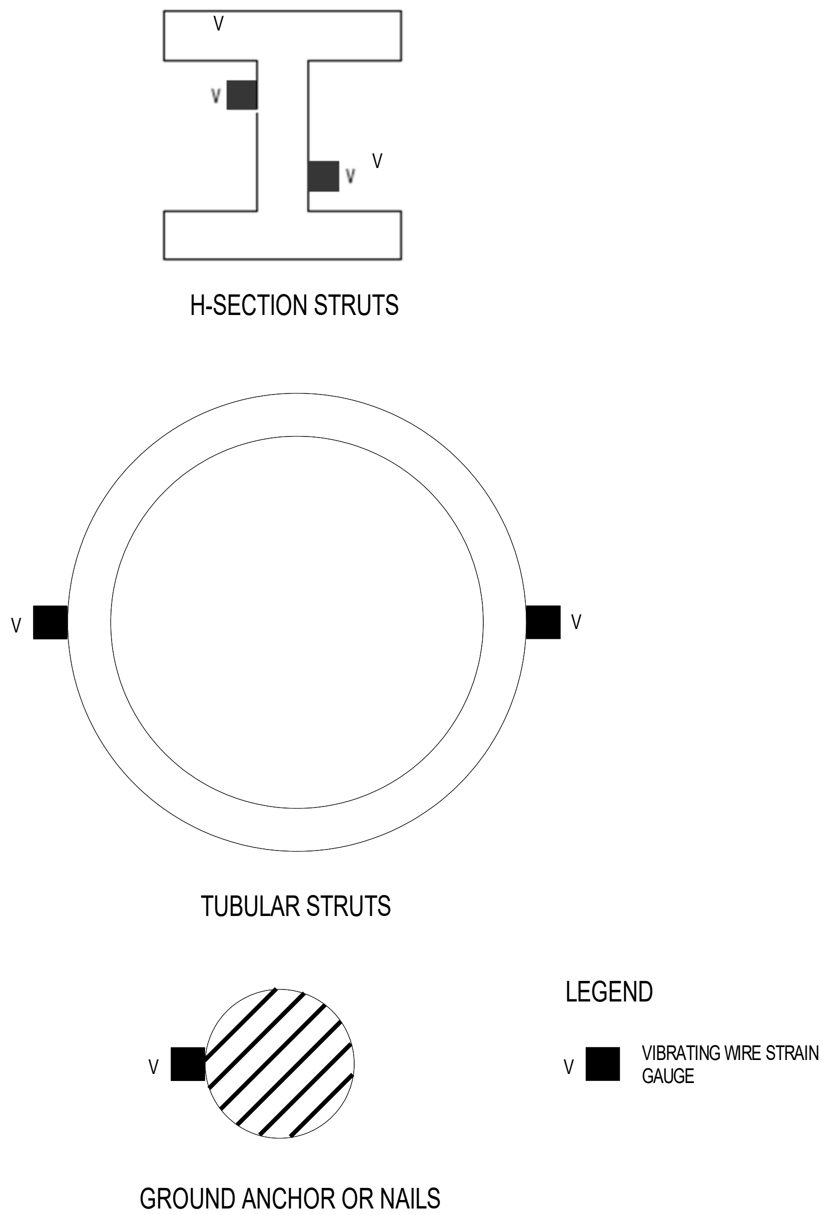
Note: The distance between the tip of subsurface instruments and tunnel external surface shall be at least one tunnel diameter.

Figure 19.6 – Monitoring Arrays for Tunnels where Any Part of the Face is in Kallang or Tekong Formation Soil



Note: The distance between the tip of subsurface instruments and tunnel external surface shall be at least one tunnel diameter.

Figure 19.7 – Arrangement of Strain Gauges for Struts, Bar Anchors and Nails



CHAPTER 20

IMPACT ASSESSMENT OF BUILDINGS AND UTILITIES

20.1 GENERAL

Both temporary and permanent works shall be designed to ensure that ground movements are kept to an absolute minimum. In addition, it shall be noted that the use of proven construction techniques and good workmanship are essential in minimising ground loss and water table drawdown.

20.2 PREDICTION OF GROUND MOVEMENTS

20.2.1 Ground Movements Due to Bored Tunnelling

20.2.1.1 Bored tunnelling work will generally produce a settlement trough which is related to a Gaussian distribution curve. The settlement trough for bored tunnelling works obtained by Peck (1969) and O'Reilly and New (1982) shall be used where there is no major loss of ground at the face and where there is little or no consolidation settlement.

20.2.1.2 For a single tunnel, immediate surface settlement, Sv, shall be determined from the relationships:

$$S_v = S_{max} \exp\left[-\frac{y^2}{2i^2}\right]$$

Where,

S_{max} is the max. settlement above the tunnel

y is the horizontal distance to the tunnel

i is the horizontal distance from the tunnel centre line to the point of inflexion on the settlement trough

$i = K z_o$

Where,

K is a parameter, which varies between 0.5 (for clay) and 0.25 (for sand)

z_o is the depth to the centre of the tunnel

$$S_{max} = \frac{0.0031\ VD^2}{i}$$

Where,

D is the excavated diameter of the tunnel

V is the volume loss expressed as a percentage of the excavated tunnel face area

20.2.1.3 Typical values for tunnels up to 6.6m excavated diameter based on the previous experience for the various ground condition are shown in Table 20.1.

Table 20.1: Typical Values for K and V

Ground Type	Tunnelling Method	K	V (%)
Jurong Formation, S(IV)/S(V)/S(VI)	Pressure Balance Machine	0.45	1.0
Jurong Formation, S(I)/S(II)/S(III)	Pressure Balance Machine	0.45	1.0
	Sprayed Concrete Lining	0.45	1.5
Mixed-face – Soil & Rock of Jurong Formation	Pressure Balance Machine	0.45	1.5
Fort Canning Boulder Bed, FCBB	Pressure Balance Machine	0.45	1.0
	Sprayed Concrete Lining	0.45	1.5
Bukit Timah Granite, G(IV)/G(V)/G(VI)	Pressure Balance Machine	0.45	1.0
Bukit Timah Granite, G(I)/ G(II)/G(III	Pressure Balance Machine	0.45	1.0
	Sprayed Concrete Lining	0.45	1.5
Mixed-face – Soil & Rock of Bukit Timah Granite	Pressure Balance Machine	0.45	3.0

Table 20.1: Typical Values for K and V (Cont'd)

Ground Type	Tunnelling Method	K	V (%)
Old Alluvium, OA	Pressure Balance Machine	0.45	1.0
	Sprayed Concrete Lining	0.45	1.5
Kallang Formation, KF	Pressure Balance Machine	0.5	3.0
Mixed-face of Kallang & Other Formation	Pressure Balance Machine	0.45	3.0

20.2.1.4 Values for tunnelling using pressure balance machines are for tunnels where sufficient pressure was maintained in the head to minimise movement at the face, and where simultaneous tail void grouting was carried out.

20.2.1.5 Volume loss indicated is for the purpose of impact assessment when all necessary measures to mitigate any potential risks of collapse or sink holes especially in mixed-face conditions are in place.

20.2.1.6 For locations of stoppages, higher volume loss shall be proposed for the acceptance of the Engineer.

20.2.1.7 Consolidation settlements shall be assessed and superimposed on the settlements calculated as described above. Settlement contour plans along the full route of the tunnels shall be prepared.

20.2.1.8 The horizontal displacements induced by tunnelling may be approximated by assuming that the displacement vectors of the ground are directed towards the tunnel axis such that horizontal ground displacement is $S_h = \left(\frac{x}{z_0}\right) S_v$ (Mair and Taylor, 1997).

20.2.1.9 Sub-surface settlements may be approximated using the Gaussian distribution curve and method proposed by Mair, Taylor and Bracegirdle (1993).

20.2.2 Ground Movements Due to Excavations

20.2.2.1 The prediction of the ground movements due to excavations shall allow for: -

a) The installation and (where appropriate) removal of the support system.

b) Movements during excavation.

c) Consolidation settlements.

d) The effects of grouting, piling, soil improvement, or any other measure required for the Works, which could cause ground movement.

20.2.2.2 Numerical modelling can give misleading results for the development of ground movements outside the excavation unless the change of ground stiffness with strain is properly taken into account. All predictions of ground movements based on numerical modelling shall be compared and checked against empirical methods based on previous experience of excavations in similar ground in Singapore.

20.2.2.3 Hydro-geological study shall be carried out to establish the hydraulic connectivity of ground water through permeable soil or rock layers and determine how the ground water connectivity will influence the extent of water table draw down beyond typical zone of influence. These effects shall be considered in the estimation of consolidation settlements by carrying out seepage analysis for all excavations.

20.2.2.4 Settlement contour plans shall be prepared for the area around excavations. Horizontal movements and strains associated with excavations shall also be predicted.

20.2.3 Combined Effects

Total ground movement that could affect the structures, buildings and utilities within the influence zone shall be calculated by combining all the ground movements due to: -

a) Tunnelling and associated work, such as shaft construction, ground treatment.

b) Excavations and associated work, such as wall installation, ground treatment.

c) The ground movements due to the work of the adjacent contractor(s), in so far as these could affect buildings within the assessment zones.

d) Any other construction work within the assessment zone that is likely to be concurrent with the Works.

20.3 ASSESSMENT OF POTENTIAL IMPACT TO BUILDINGS AND STRUCTURES

20.3.1 Assessment of potential impact shall be made for all buildings and structures within the zone of influence of the works. Figure 20.1 shows the minimum zone of influence for tunnelling and deep excavation activities. However, the zone of influence shall be reviewed by the designer based on his assessment and where necessary, shall be increased accordingly.

20.3.2 The methodology for assessment of potential impact to buildings or structures is outlined in Appendix 20.1 (after Fok et al 2012).

20.3.3 Visual Inspection of Existing Structures and Buildings

Visual inspections of buildings and structures within the influence zone of underground construction shall be conducted at the start of impact assessment studies. This would include the identification of structural defects, signs of structural distress, unusual structural deformations, material deterioration, water seepage, and adverse or unauthorised loading conditions and/or unauthorised structural modifications. The inspection report shall include photographic records of observations and an assessment on the existing condition of the building. Appendix 20.2 provides guidance on the classification of existing condition of a building.

20.3.4 Classification of Category of Damage

20.3.4.1 The classification of impact/damage associated with various degrees of severity and having particular reference to ease of repair of plaster and brickwork, is shown in Appendix 20.3. This was developed through the work of Burland, Broms and De Mello (1977) and follows the classification system adopted by the Building Research Establishment.

20.3.4.2 The onset of cracking in a building has been found to be closely related to the development of tensile strains in the building. By verifying with actual case histories and full-scale tests, Burland, Broms and De Mello (1977) and Boscardin and Cording (1989) showed that the category of building impact can be related to the limiting tensile strains and this relationship is shown in Table 20.2.

Table 20.2: Damage Category for Masonry Buildings

Category of damage	Normal degree of severity	Limiting tensile strain ($\in_{lim}$) (%)
0	Negligible	0 – 0.05
1	Very Slight	0.05 – 0.075
2	Slight	0.075 – 0.15
3	Moderate	0.15 – 0.3
4 to 5	Severe to Very Severe	> 0.3

20.3.4.3 It should be highlighted that the classification in Appendix 20.3 and Table 20.2 was developed primarily for brickwork or blockwork and stone masonry, these could also be adopted for other forms of cladding.

20.3.4.4 Due account could be taken of any stiffness in the buildings to resist ground deformations.

20.3.4.5 The actual impact on reinforced concrete structures due to induced ground movements can only be ascertained using structural analysis and checked against the structural capacities of the key load-carrying elements. Nevertheless, the potential impact induced onto reinforced concrete structures may be approximated to the impact induced onto their cladding and walls, and can thus be assessed using a similar methodology outlined above using tensile strains but with a slight modification on the E/G ratio (after Mair, Taylor and Burland, 1996).

20.3.4.6 In addition, a serviceability performance review shall be carried out. The factors that could be taken into account in the serviceability performance review shall include the following but not be limited to: -

- Structural continuity
- Foundation system
- Soil-structure interaction
- Basement configuration and waterproofing systems
- Cladding system
- Orientation to underground construction alignment
- Existing conditions
- Construction sequence

20.3.5 Staged Approach to Risk Assessment

20.3.5.1 For buildings on shallow foundations, the staged assessment approach outlined in Mair, Taylor and Burland (1996), shall be used: -

- Stage 1 Preliminary Assessment

  A building falling within a settlement zone of less than 10mm and experiencing differential settlement of a slope less than 1:500 can be considered to have a negligible risk of impact. No further assessment is required.

- Stage 2 Assessment based on Tensile Strain

  This is based on estimating the maximum tensile strain induced in the building by idealising the building into a beam, and then checking the induced maximum tensile strain against the limiting tensile strain corresponding to various building impact categories as shown in Table 20.2. This approach assumes that the building has no stiffness but conforms to the greenfield settlement profile, and will deflect according to the deflection ratios and horizontal strains corresponding to the greenfield. This is a conservative assumption as the inherent stiffness of buildings will reduce both the deflection ratio and horizontal strains. The foundation type and actual stiffness of the buildings can be considered where applicable. Buildings assessed to have negligible to slight degree of severity as defined in Table 20.2 are considered to be at low risk of impact. No further assessment is required.

- Stage 3 Detailed Evaluation

  For buildings assessed to have a moderate and higher degree of severity, detailed evaluation is to be undertaken. This would require (but not limited to) evaluating the structural details of the building, giving full consideration of the sequence and method of construction and considering soil-structure interaction effects.

20.3.5.2 Following the detailed evaluation, mitigation and/or protective measures shall be provided for buildings that remain in the moderate risk of damage or higher.

20.3.5.3 The assessment for every building and structure within the influence zone shall be summarised, using the format shown in Figure 20.2.

20.3.6 Buildings on Pile Foundations

20.3.6.1 Where a building or structure is founded on piles and located within the influence zone, lateral and vertical ground movements caused by an adjacent tunnelling or excavation could induce lateral and vertical forces onto the piles. An assessment on the adequacy of the pile capacity shall be carried out by estimating the additional forces and bending moments induced by such ground movements.

20.3.6.2 The potential for displacements at the top of the piles exceeding those calculated for 'greenfield' conditions shall be evaluated and the effect on the building or structure assessed. Specifically, considerations shall be made to estimate individual pile settlement with regards to tunnelling construction below pile toes. Based on the studies by Kaalberg et al. (2005) and Selematas et al. (2005), it is proposed to classify pile settlement according to the position of the pile toe relative to the tunnel axis and as illustrated in Figure 20.3. For piles with toes directly above the tunnel (Zone A), it was found that piles settled more than the ground surface due to a reduction in their base load and an increased mobilisation of shaft friction during tunnelling. Piles with toes that are within Zone B settled more or less by the same amount as the ground surface, whereas piles with toes within Zone C settled less than the ground surface. It is important to highlight that the boundaries of Zone B were simplified by drawing two straight lines from the tunnel axis. The extent of Zone B would depend on shearing resistance of the soil and the volume loss induced during tunnelling.

20.3.6.3 If a building or structure is founded on piles located within more than one zone, the effect on the building or structure due to differential movement between the piles shall also be evaluated.

20.3.6.4 For high rise buildings, particular attention has to be given to assess the allowable tilt arising from pile head movements and rotations.

20.3.7 Buildings on Mixed Foundations and Sensitive Structures

20.3.7.1 For any building or structure that is identified as being on mixed foundations, a Stage 3 Detailed Evaluation shall be carried out, taking into account the nature of the building or structure and the foundations.

20.3.7.2 For buildings that are classified as monuments and conservation buildings, elements and components of the building may have deteriorated, such as the mortar holding the tiles in place and the mortar between masonry materials becoming more brittle with time. These are classified as sensitive structures, and a Stage 3 Detailed Evaluation should be carried out taking into account the tolerable movements allowed by the finishes, claddings, and masonry.

20.3.8 Buildings in Poor Existing Condition

The existing condition of the building shall be incorporated into the building impact assessment works, see table in Appendix 20.3. For buildings that are found to be in poor condition after visual inspection, a Stage 3 Detailed Evaluation should be carried out in the assessment of building impact due to excavation and tunnelling activities.

20.3.9 Alternative Method to assess Sensitive Structures

20.3.9.1 The method can be done using a risk-based, index system to the tensile strain method. Table 20.3 shows the categories of sensitivity indices for local application.

Table 20.3: Categories of Sensitivity Indices to Incorporate Existing Structural Form, Heritage Features and Existing Condition (after Goh and Mair, 2014)

Categories of Sensitivity Indices	Structural Form	Heritage Features	Existing Condition
	Sensitivity of the structure to ground movements and interaction with adjacent buildings	**Sensitivity to calculated movement of particular features within the building**	**Factors which may affect sensitive structures to further movements**
0	Similar to buildings for which the Burland et al assessment was formulated	No particular sensitive features	Good or Satisfactory (based on the visual inspection guidance given in the Appendix 20.2)
1	Buildings with some sensitive structural features in the movement zone (e.g. existing cracks where further movements are likely to concentrate)	Brittle finishes, e.g. faience or tight-jointed stonework, which are susceptible to small structural movements and difficult to repair invisibly	Poor condition – may change the structural behaviour in cases of additional movements. Evidence of on-going movements that caused existing damage, but this relates only to serviceability concerns and unlikely to affect the structural stability and integrity to any part of the building
2	Buildings which by their structural form will tend to concentrate all their movements in one location (e.g. buildings on mixed foundations, shophouses on shallow foundations abutting each other, long wall without movement joints)	Finishes which if damaged will have a significant effect on the heritage value of the building	Poor condition – especially if existing damage are likely to affect the structural stability and integrity to any part of the building, and also if very small movements could lead to significant impact

20.3.9.2 The category of damage estimated using the tensile strain method shall be added together with all the individual sensitivity indices for structural form, heritage features, and existing condition, respectively to give a final category of damage.

20.4 ASSESSMENT OF IMPACT TO UTILITIES

20.4.1 An impact assessment for every utility within the zone of influence of the works shall be carried out. The allowable values for settlement, deformation, joint rotation, joint slip, or such other criteria as agreed with the utility agency shall be established. Assessments shall be carried out to ensure that the serviceability of the utility shall not be affected during and after the Works. Where necessary, trial pits shall be carried out to confirm the nature of the utility and joint spacing.

20.4.2 Particular attention shall be given to the connection of the pipes, as outlined by Attewell, Yeates and Selby (1996). For cast iron pipes the methodology of Bracegirdle et al (1996) shall be followed.

20.4.3 The assessment for every utility within the zone of influence of the works shall be tabulated. The summary shall include details of the type of utility, nature, joint type and spacing, and the allowable and predicted values for settlement, deformation, joint rotation, joint slip, or such other criteria as are agreed with the utility agency.

20.5 PROTECTIVE WORKS

20.5.1 For all structures where the predicted degree of severity of settlement impact is “moderate” or above, protective works shall be designed and installed with the aim to mitigate the impact to “slight” category and below.

20.5.2 Where a building has historical or other significance, as stated in the Particular Specification, protective works shall be designed and installed with the aim to mitigate the induced damage to “very slight” category and below.

20.5.3 For reinforced/prestressed concrete structures where the predicted serviceability limit state is exceeded, protective works shall be designed and installed to conform to the serviceability limit state criteria.

20.5.4 Protective works shall be in the form of underpinning, ground improvements, compaction or compensation grouting, jacking or building strengthening or some combination of these or such other means to the acceptance of the Engineer. The protective works shall not induce any adverse impact on the building during implementation and removal. Detailed sequences and methods shall be developed with monitoring provisions to verify that the building behaves as intended.

20.5.5 Where any of the predicted values for a utility exceeds the allowable value, protective works shall be designed and implemented. Protective works shall be in the form of supporting, underpinning, ground improvements, compaction or compensation grouting, jacking or such other means to the acceptance of the Engineer.

20.5.6 If a building, structure or utility is likely to be affected by ground movements due to the work of two or more contractors, and unacceptable damage is predicted, appropriate protective measures shall be co-ordinated amongst various designers.

References

Attewell, P.B., Yeates, J. and Selby, A.R. 1986. Soil movements induced by tunnelling and their effects on pipelines and structures. Blackie and Sons Ltd., London, UK.

Boscardin, M.D. and Cording E.J. 1989. Building response to excavation-induced and settlement. Journal Geotechnical Engineering, ASCE, Vol 115:1(1), pp. 1-21.

Bracegirdle, A. Mair, R.J., Nyren, R.J. and Taylor, R.N. 1996. A simple methodology for evaluating the potential damage to buried cast iron pipes from ground movement arising from tunnelling. Int. Symp. on Geotechnical Aspects of Underground Construction in Soft Ground, London.

Burland, J.B., Broms, B.B. and De Mello, V.F.B. 1977. Behaviour of foundations and structures. Proceedings of the 9th Int Conference of Soil Mechanics and Foundation Engineering, Tokyo.

Fok, P., Neo, B.H., Goh, K.H. and Wen, D. 2012. Assessing the impact of excavation-induced movements on adjacent buildings, The IES Journal Part A: Civil & Structural Engineering, Vol. 5(3), pp. 195-203.

Goh, K.H. and Mair, R.J. 2014. Incorporating the existing condition of buildings when assessing impact of underground construction, Proc. of conference Underground Singapore 2014, pp.315-323.

Jacobsz, S.W., Standing, J.R., Mair, R.J., Soga, K., Hagiwara, T. and Sugiyama, T. 2001. The effects of tunnelling near driven piles in sand. Proc. of Asian Regional Conf. on Geotechnical Aspects of Underground Construction in Soft Ground, Shanghai, pp. 29-35.

Kaalberg, F.J., Teunissen, E.A.H., Tol, A.F. van and Bosch, J.W. 2005. Dutch Research on the impact of shield tunnelling on pile foundations. Proc. of the 5th Int. Symposium on Geotechnical Aspects of Underground Construction in Soft Ground, Amsterdam, pp.123-131. Rotterdam: Balkema.

Mair, R.J, Taylor, R.N. and Burland, J.B. 1996. Prediction of ground movements and assessment of risk of building damage due to bored tunnelling. Int. Symp. on Geotechnical Aspects of Underground Construction in Soft Ground, London.

Mair, R.J. and Taylor, R.N. 1997. Bored tunnelling in the urban environment: state-of-the-art report and theme lecture. Proc. 14th International Conference on Soil Mechanics and Foundation Engineering, Hamburg, 6-12 September 1997, 4: 2353–2385. Rotterdam: Balkema.

Mair, R.J., Taylor, R.N. and Bracegirdle, A. 1993. Subsurface settlement profiles above tunnels in clays. Géotechnique, Vol. 43(2), pp. 315–320.

New, B. M. and O'Reilly, M.P. 1991. Tunnelling induced ground movements: predicting their magnitude and effect. 4th Int. Conf. on Ground Movements and Structures, Cardiff.

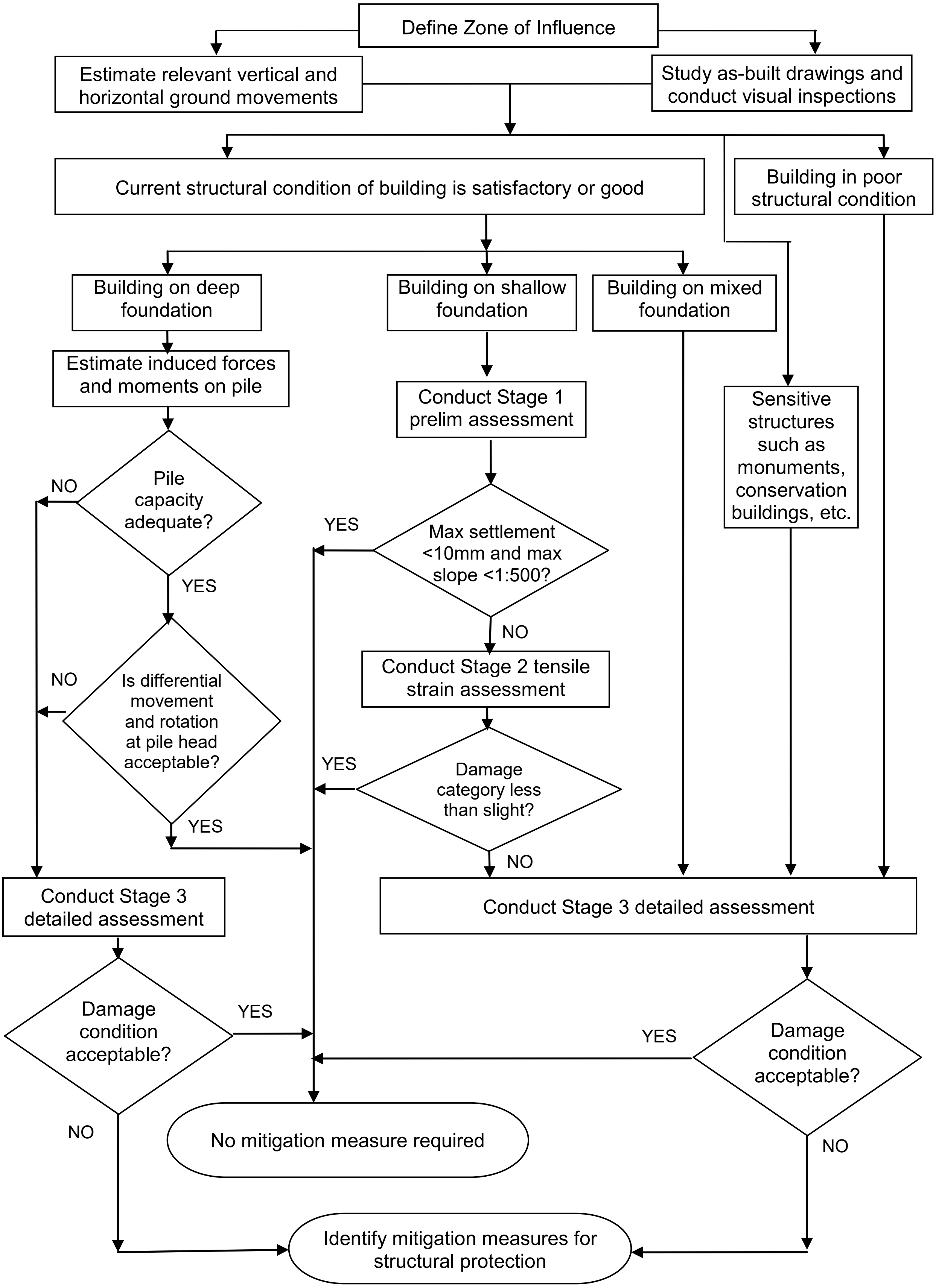
O'Reilly, M.P. and New, B.M. 1982. Settlements above tunnels in the United Kingdom - their magnitude and prediction. Tunnelling '82, Inst. Mining & Metallurgy, London, pp. 173-188.

Peck, R.B. 1969. Deep excavation and tunnelling in soft ground. State of the Art Report. 7th International Conference on Soil Mechanics and Foundation Engineering, Mexico City, pp. 225-290.

Selemetas, D., Standing, J.R. and Mair, R.J. 2005. The response of full-scale piles to tunnelling. Proc. of the 5th Int. Symposium on Geotechnical Aspects of Underground Construction in Soft Ground, Amsterdam, pp.763-769, Rotterdam: Balkema.

Appendix 20.1

Methodology for Assessment of Potential Damage to Buildings or Structures (after Fok et al., 2012)



Appendix 20.2

Guidance to classify existing condition of buildings from visual inspections (after Goh and Mair, 2014)

Classification of overall building condition	**Observations on structural elements**	**Adapted from BRE Digest 251 (1995)**	
		Classification of visible damage to walls (brick, blockwork) with particular reference to ease of repair in italics	**Classification of visible damage on ground floor slab settlement**
Good	No sign of any structural damage/ distress and material deterioration on visible structural elements	Hairline cracks of less than about 0.1mm	Hairline cracks between floor and skirtings
		Fine cracks which can *be treated easily using normal decoration*. Damage generally restricted to internal wall finishes; cracks rarely visible in external brickwork. Typical crack widths* up to 1 mm.	Settlement of the floor slab, either at a corner or along a short wall, or possibly uniformly, such that a gap opens up below skirting boards which can be masked by resetting skirting boards. No cracks in walls. No cracks in floor slab, although there may be negligible cracks in floor screed and finish. Slab reasonably level. Typical gap** up to 6mm.
Satisfactory	No sign of structural distress on visible structural elements, but some structural cracks and material deterioration (e.g. concrete spalling, corrosion of steel reinforcements) are observed. Generally, the defects are unlikely to affect structural stability and integrity of any part of the building	*Cracks easily filled. Recurrent cracks can be masked by suitable linings*. Cracks not necessarily visible externally; *some external repointing may be required to ensure weather-tightness.* Doors and windows may stick slightly and *require easing and adjusting*. Typical crack widths* up to 5 mm.	Larger gaps below skirting boards, some obvious but limited local settlement leading to a slight slope of floor slab; gaps can be masked by resetting skirting boards and some local rescreeding may be necessary. Fine cracks appear in internal partition walls which need some re-decoration; slight distortion in door frames so some ‘jamming’ may occur, necessitating adjustment of doors. No cracks in floor slab although there may be very slight cracks in floor screed and finish. Slab reasonably level. Typical crack widths* up to 1mm and gap** up to 13mm.
		Cracks which *require some opening up and can be patched by a mason. Repointing of external brickwork and possibly a small amount of brickwork to be replaced*. Doors and windows sticking. Service pipes may fracture. Weather-tightness often impaired. Typical crack widths* are 5 to 15 mm, or several of 3mm cracks.	Significant gaps below skirting boards with areas of floor, especially at corners or ends, where local settlements may have caused slight cracking of floor slab. Sloping of floor in these areas is clearly visible; (slope approximately 1 in 150). Some disruption to drain, plumbing or heating pipes may occur. Damage to internal walls is more widespread with some crack filling or replastering of partitions necessary. Doors may have to be refitted. Inspection reveals some voids below slab with poor or loosely compacted fill. Typical crack widths* up to 5mm and gap** up to 19mm.

Appendix 20.2

Guidance to classify existing condition of buildings from visual inspections (after Goh and Mair, 2014) (Cont'd)

Classification of overall building condition	**Observations on structural elements**	**Adapted from BRE Digest 251 (1995)**	
		Classification of visible damage to walls (brick, blockwork) with particular reference to ease of repair in italics	**Classification of visible damage on ground floor slab settlement**
Poor	Signs of structural distress and structural damage on visible structural elements. These defects are likely to affect the structural stability and integrity to any part of the building,	Extensive damage which *requires breaking-out and replacing sections of walls*, especially over doors and windows. Windows and door frames distorted, floor sloping noticeably***. Walls leaning or bulging noticeably, some loss of bearing in beams. Service pipes disrupted. Typical crack widths* are 15 to 25 mm, but also depends on number of cracks.	Large, localised gaps below skirting boards; possibly some cracks in floor slab with sharp fall to edge of slab; (slope approximately 1 in 500 or more). Inspection reveals voids exceeding 50 mm below slab and/or poor or loose fill likely to settle further. Local breaking out, part refilling and relaying of floor slab or grouting of fill may be necessary; damage to internal partitions may require replacement of some bricks or blocks or relining of stud partitions. Typical crack widths* range between 5 to 15mm but may also depend on number of cracks, and gap** up to 25mm.
		Structural damage which *requires a major repair job, involving partial or complete rebuilding*. Beams lose bearing, walls lean badly and require shoring. Windows broken with distortion. Danger of instability. Typical crack widths* are greater than 25 mm, but depends on number of cracks.	Either very large, overall floor settlement with large movement of walls and damage at junctions extending up into 1st floor area, with possible damage to exterior walls, or large differential settlements across floor slab. Voids exceeding 75mm below slab and/or very poor or very loose fill likely to settle further. Risk of instability. Most or all of floor slab requires breaking out and relaying or grouting of fill; internal partitions need replacement. Typical crack widths* greater than 15mm but may also depend on number of cracks, and gap** greater than 25mm.

* Crack width is one factor in assessing category of damage and should not be used on its own as a direct measure of it.

** Gap refers to space usually between the skirting and finished floor, caused by settlement after making appropriate allowance for discrepancy in building, shrinkage, normal bedding and the like.

*** For floors and walls, local deviation of slope from the horizontal or vertical of more than 1/100 will normally be clearly visible. Overall deviations in excess of 1/150 are undesirable.

Appendix 20.3

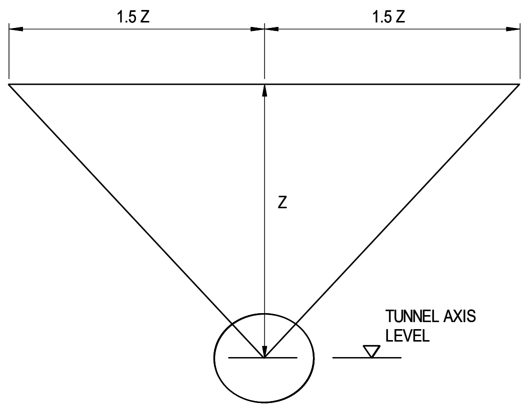
Classification of Visible Damage to Walls with Particular Reference to Ease of Repair of Plaster and Brickwork or Masonry.
(after Mair et al. 1996)

Category of damage	**Normal degree of severity**	**Description of typical damage (Ease of repair is underlined) Note: Crack width is only one factor in assessing category of damage and should not be used on its own as a direct measure of it.**
0	Negligible	Hairline cracks less than about 0.1mm.
1	Very Slight	Fine cracks which are easily treated during normal decoration. Damage generally restricted to internal wall finishes. Close inspection may reveal some cracks in external brickwork or masonry. Typical crack widths up to 1mm.
2	Slight	Cracks easily filled. Re-decoration probably required. Recurrent cracks can be masked by suitable linings. Cracks may be visible externally and some repointing may be required to ensure weathertightness. Doors and windows may stick slightly. Typical crack widths up to 5mm.
3	Moderate	The cracks require some opening up and can be patched by a mason. Repointing of external brickwork and possibly a small amount of brickwork to be replaced. Doors and windows sticking. Service pipes may fracture. Weathertightness often impaired. Typical crack widths are 5 to 15mm or several greater than 3mm.
4	Severe	Extensive repair work involving breaking-out and replacing sections of walls, especially over doors and windows. Windows and door frames distorted, floor sloping noticeably[1]. Walls or building leaning[1] noticeably, some loss of bearing in beams. Service pipes disrupted. Typical crack widths are 15 to 25mm but also depends on the number of cracks.
5	Very Severe	This requires a major repair job involving partial or complete rebuilding. Beams lose bearing, walls lean badly and require shoring. Windows broken with distortion. Danger of instability. Typical crack widths are greater than 25mm but depends on the number of cracks.

[1] Note: Local deviation of slope, from the horizontal or vertical, of more than 1/100 will normally be clearly visible. Overall deviations in excess of 1/150 are undesirable.

Figure 20.1 – Minimum zone of influence for tunnelling works and excavation activities after Clough and O'Rourke (1990) and Fok et al. (2012)

1. TUNNELS



Note: Figure is for tunnel with 6.6m diameter. For deeper tunnel beyond 30m, influence zone may be adjusted to 2 x 1.35Z subject to the Engineer's approval.

2. EXCAVATIONS

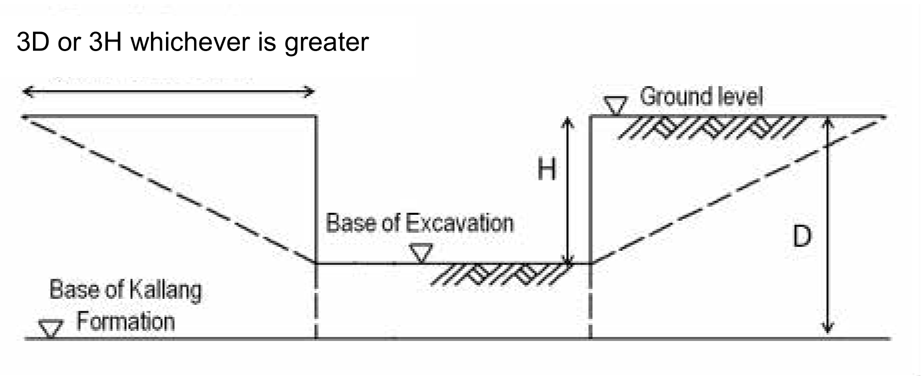


Figure 20.2 – BUILDING IMPACT ASSESSEMENT SUMMARY SHEET

Project:

Contract Number:

Name of Building:

Address:

Description of Structure:

PHOTO

Drawings available YES/NO

Description of Foundations

Drawings available YES/NO

Result of Preliminary Assessment
Maximum settlement:
Maximum slope:
Second stage assessment required YES/NO

Result of Second Stage Assessment
Maximum settlement:
Maximum slope:
Maximum tensile strain:

Detailed evaluation required YES/NO

Attach results of detailed evaluation when required

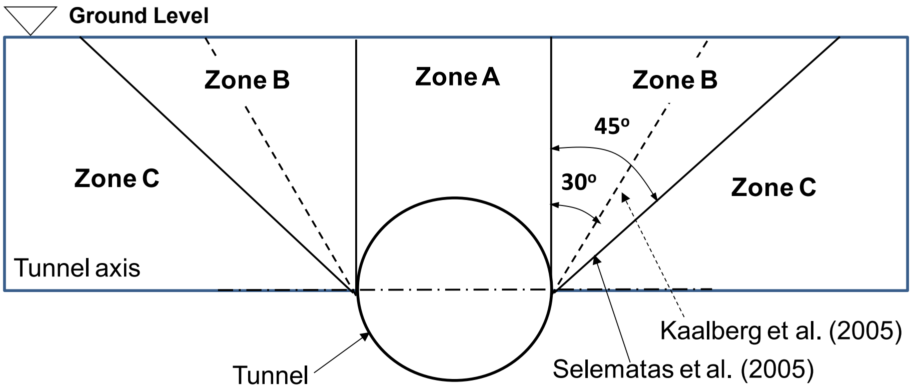
Protection measures needed YES/NO

Protection measures proposed:

Figure 20.3 – Idealised zones of pile settlement during tunnelling with definition of Zone "A", "B" and "C" after Jacobsz et al (2001), Kaalberg et al (2005) and Selematas, Standing and Mair (2005)

$$R = \frac{\text{Pile Head Settlement}}{\text{Ground Surface Settlement}}$$

Zone A – R > 1
Zone B – R = 1
Zone C – R < 1



CHAPTER 21

LIGHTING SYSTEM

21.1 PUBLIC STREET LIGHTING

21.1.1 General

The design of lighting shall be based on the latest edition of the following publications:

(a) BS 5489 (British Standard)

(b) CIE 115 (Technical Report of Commission Internationale de L'Eclairage)

Road surface shall be taken as Class R3 road (Asphalt CIE R3). The design shall also comply with the applicable Codes, Regulations, Standards and relevant Authorities.

For new major roads and expressways, IP66 high performance lantern mounted on 10 meter pole height with 250 watts High Pressure Sodium Vapour (HPSV) lamp (of min 33,000 lumens) shall be used. The distance between each lamp pole required shall be minimum 45m, to be determined in accordance to the lighting level requirement. The lantern shall be provided with multiple adjustments to have variable light distribution to suit the road conditions. The total light output ratio shall equal or exceed 75%.

21.1.2 Lighting Level Requirements

The designed lighting levels (average illuminance at floor level) for expressway and major road is 20 lux. Minor and residential road is 10 lux.

The overall uniformity of the light distribution on the road shall be 0.30 for all categories of road. The uniformity is defined as the ratio of minimum illuminance to the designed average illuminance.

The luminaries shall meet the following requirements:

(a) Luminaires shall be high performance types with tampered glass bowls.

(b) Each luminaire shall be adjustable for at least full cut-off and semi cut-off light distribution as per CIE (International Commission on Illumination) definitions.

(c) Luminaires shall be completed with integral control gears and shall be suitable for use on a supply voltage of 230 volts ±6%, 50 Hz.

(d) Luminaires shall be suitable for use with 70W (minimum 6,600 lumens), 150W (minimum 17,500 lumens), 250W (minimum 33,000 lumens) or 400W (minimum 56,000 lumens) tubular clear high pressure sodium vapour lamps.

(e) The mounting heights of the luminaries shall be between 6 m (minimum) to 10m depending on the actual requirements.

(f) The mounting arrangement and wind resistant area of a luminaire shall be such as to withstand a wind-speed of 100 km per hour with a factor of safety of 5.

21.1.3 Works in Conjunction with Lighting

The following works shall be carried out:

(a) Mounting details including all data, calculations, imposed loads and forces and dimensional drawings for the foundations required (piling if necessary) for the poles shall be endorsed by a registered Professional Engineer.

(b) The soil bearing capacity at the site shall be ascertained so that the foundations can be correctly designed.

(c) Base plates, holding-down bolts etc. shall be provided for the installation of street lighting. Heavy-duty UPVC pipes comply to SS141 Class B with pull-wire shall be provided for the underground cables.

(d) All outgoing underground cables from street lighting control box shall be supplied and laid by LTA.

(e) The developer shall carry out co-ordination and interfacing of incoming supply cabling work to street lighting control box with SP PowerGrid Ltd.

21.2 VEHICULAR UNDERPASS LIGHTING

21.2.1 General

Road surface shall be taken as Class R3 road (Asphalt CIE R3). Uniformity in the underpass shall not be less than 0.8.

The design of lighting shall be based on the latest edition of the following publications:

(a) BS 5489 (British Standard)

(b) CIE 88 (Technical Report of Commission Internationale de L'Eclairage)

The design shall also comply with the applicable Codes, Regulations, Standards and relevant Authorities.

The underpass lighting shall be provided with electronic ballasts for dimming to night time lighting level. The night time lighting level shall be in accordance with CIE88.

Dimming of the fluorescent luminaries shall be continuous.

The lighting system shall be controlled via programmable timers with RS232 interface. The timers shall be supplied inclusive of a bypass switch.

21.2.2 Emergency Lighting

In the event of power failure, the lighting level shall be in accordance with CIE88 and shall be maintained for a minimum 1 hour of operations.

21.2.3 Luminaires Requirements

The luminaries shall meet the following requirements:

(a) All luminaires complete with all accessories shall be of a type specially designed for use in vehicular underpass.

(b) Luminaires shall comply with BS 4533 and IEC Degrees of Protection IP 65 including requirements in jet proofing, thermal testing and dust proofing.

All underpass lighting sub-circuits shall consist of fire resistant, low smoke, halogen free armoured cables.

Design calculations to justify on the adequacy of the support system with details of all equipment and cables total weight in worst case shall be endorsed by a Professional Engineer.

21.3 TUNNEL LIGHTING

21.3.1 General

The design of lighting shall be based on the latest edition of the following publications:

(a) Road lighting Part 7 Code of Practice for the lighting of tunnels and underpasses – BS 5489 : Part 7 (British Standard)

(b) Guide for the lighting of road tunnels and underpasses – CIE 88 (Technical Report of Commission Internationale de L'Eclairage)

(c) Permanent International Associations of Roads Congresses – Report to 18th, 19th and 20th Congresses on Road Tunnels – 1987, 1991 and 1995

21.3.2 Design Parameters

In general, the recommendations of the latest edition of CIE88/BS 5489 shall be used for detail design work. The following parameters are to be considered in the design:

(a)	Design Speed	:	The design speed used in the lighting calculations shall be subject to the approval of the Authority.
(b)	Stopping sight distance	:	The stopping sight distance shall be based on the recommendations of CIE 88/BS5489.
(c)	The access zone luminance (L20)	:	The access zone luminance shall be based on recommendations in CIE88/BS5489 and subject to site investigations.
(d)	Switching control	:	Luminance meters shall be provided at entrance portals to adjust the entrance zone lighting level in accordance with the actual access zone luminance.
(e)	Lamplife and maintenance	:	The design shall consider the optimisation of lamplife and maintenance.

21.3.3 Glare Control

Glare control as Threshold Increment (TI) set out within CIE88/BS 5489 shall be considered within the design and selection of luminaire and type of light source. The TI within the tunnel shall not exceed 15%.

21.3.4 Emergency Lighting

In the event of power failure, backup provisions shall be provided for the tunnel lighting system by means of both Uninterruptible Power Supply (UPS) and standby generator systems.

The UPS shall be provided to ensure that there is no interruption of the tunnel lighting when transferring to standby generator supply in the event of power failure. A minimum full load backup capacity of half-hour shall be provided.

Emergency electrical power by diesel generators shall be provided for road tunnel lighting. A day tank of 700 litres shall be provided and a manual and/ or automatic fuel transfer system to give at least 6 hours of full load emergency back-up shall be provided. Additional fuel storage capacity to meet operation response time shall be considered in planning the capacity of the fuel system. The storage tank shall be housed above ground level unless there are site constraints. In general, the system shall comply with SS CP 31 and SS CP 40.

APPENDIX A

DESIGN CHECK LIST

The Contractor shall ensure that his design as shown on all Drawings and Contract documents is complete.

The check list as shown on the following pages is compiled from obvious omissions in previous MRT phases. The list is not meant to be exhaustive and it is the Contractor's duty to ensure that he has a complete design for construction.

A STATIONS - GENERAL

1. Access catladder - Provide approved standardised catladders to underplatform voids, vent shafts, hatches, cable towers, E&M equipment, etc.

2. Platform endstairs - Provide safe platform endstairs for use during emergency situation, i.e adequate width, preferably 900 mm, but not less than 600 mm, and bottom riser not more than 170 mm.

3. Floor hatch cover - All floor hatch cover should be flush with the floor and protected against damage by heavy E & M equipment movement.

4. Maintenance Platform - Provide adequate platform with railings/catladders for maintenance access to E&M equipment, louvres, valves, water tanks, switches, etc at high level.

5. Platform end walkways - E&M equipment mounted on the walls should not cause the platform end walkways to have inadequate width for emergency egress.

6. Escalators - The railing balustrade design should tie in with the escalator to prevent children from falling.

   Slope surface between two escalators need some means to deter people from sliding down.

   Provide deterrent triangles at beams/slabs that are near the escalators edge.

   The gap between the escalator handrail and adjacent handrail should be at least 80 mm apart.

   Provide safety switch at roller shutter gate entrance so that the escalators will only move when the shutter gate is fully opened.

7. Channel Gratings - provide gratings over channels/drains where access is needed.

8. Taps and Drain Pipes - All discharge points to water taps and aircon drain pipes shall be directly over or near floor traps.

   No water tap is allowed to be installed inside the distribution board room or in other rooms where there are electrical accessories. It should be located at least 2 m away.

9. PUB valve chambers - All PUB incoming valve chambers are to be designed with drainage provision and meter reading facility.

10. Movement Joints - Ensure that there are no movement joints above plant rooms to eliminate risk of leakage of water into these rooms.

11. Cover for incoming PUB meter and control valve - Ensure that cover is light enough for regular inspection and emergency usage and at the same time, secure enough to prevent the cover from pilferage.

B UNDERGROUND STATIONS

1. Buffer Area Railings - Provide handrailings on buffer area outside the traction substation to 5 m from the station screen door end return.

2. Halon Cylinder Storage - Provide proper fire-rated enclosure for halon cylinder storage.

3. Doors with direct access to track - Provide safety bar and warning sign for doors that provide direct access to trackside from station rooms.

4. CD airfilters - CD station airfilters in rooms should not obstruct the opening of the door access.

5. Cooling tower rooms - Design for adequate drainage facilities to prevent flooding of rooms due to backflow of water from cooling tower.

C ELEVATED STATIONS

1. Utility Duct - All the utility duct wall enclosures are to be sealed for proper fire rating.

2. Ancillary Roof Access to utility ducts - The design should incorporate proper and safe access from ancillary roof area 1 to roof area 2.

3. Platform edge for substation - Handrailings are to be provided along the platform edge for Rectifier, and L.V. switch rooms in front of the entrance for on line substation.

4. Emergency escape doors - All Emergency escape doors leading to outside the station should not have a great drop in level.

5. Intermediate roof slab - Warning sign required along handrail of catwalk to deter people from steeping onto ceiling panel of concourse level.

6. Flat roof at platform level - Proper access is required for maintenance of aircon fan coils.

7. Collection trough - The track side collection trough for rainwater downpipe should be covered with a grating where appropriate.

8. Compressor rooms for air condition - Ensure that such rooms have sufficient ventilation to prevent breakdown of operation.

D DEPOT

1. Manhole access cover - The manhole access cover shall be of reasonable weight for easy handling in all buildings.

2. Ventilation Fans - provide permanent access to all high level ventilation fans in depot building.

3. Gantry Platform - Provide kick boards for gantry platform.

4. Rail Infringement - Drain grating at open position shall not infringe the rail at tracks entering the depot building.

5. Windows swing - Sliding windows should be adopted to buildings with walkways adjacent to it.

6. Volatile Store - Roller shutter without perforation shall not be used as it does not provide free air ventilation to the room.

7. Fuel Station - Concrete pavement instead of bitumen surface around the fuel station shall be adopted.

8. Upstand Kerb - All upstand kerb of cable manhole should be highlighted with black yellow strip marking.

E TUNNELS

1. Sump Pump Control Panel - Provide proper access to tunnel sump pump control panels.

2. Blast Door roller housing - Provide ramps to blast door roller housing which is above the track level.

3. Tunnel Booster Fans - provide collapse frame and maintenance access platform to the tunnel booster fan units.

4. Floating Slab Track - gap between floating slab track and tunnel wall to be backfilled.

5. Trackside Drain - trackside drain near platform steps to be covered with gratings.

F VIADUCTS

1. Low viaduct parapet on non-walkway side - Provide additional railings on the low parapet wall where there is no third rail to act as deterrent for staff safety.

2. Viaduct near station end stairs - 3 m of handrailing to be provided on parapet leading to emergency access from track to end stairs.

3. Bearings - Are they all accessible for future maintenance and replacement?

   Check at non-standard location such as cross over beams, at station etc.

4. Drainage system for viaduct - Is the system designed for movements of viaduct beams?

5. Walkways - Should have no dead ends.

6. Railings - To be provided near station at places where detrained passenger is likely to make use of for escape.

7. Vandals - Are there adequate barriers at at-grade section to prevent vandals from getting onto viaducts?

G TRACKWORKS

1. Steel grating shall be lightweight covers for tunnel sump instead of heavy concrete covers.

2. Buffer rubber (or wood) shall be fixed on buffer stops in depot.

3. Buffer stops at end of viaducts to be designed to stop train travelling at maximum speed in restricted mode of 18 km/h.

4. Difference in levels of rubber joint packer between Floating Slab units and tunnel would cause tripping hazard. Proposed maximum difference in level of 10 mm and maximum gap of 5 mm.

5. Fill up cable recess of traction jumper cables between running rails on slab track in tunnel.

6. Ballast infill between tracks and near sectionalising switches. Propose the following :-

   i) Sect. switches - 150 mm below plinth level

   ii) Track spacing 4.7 to 6 m - 200 mm thick layer of ballast

   iii) Track spacing > 6 m - blockwork kerb 150 mm high to retain ballast