ANALYSIS AND DESIGN OF REINFORCED AND PRESTRESSED-CONCRETE GUIDEWAY STRUCTURES

Reported by ACI Committee 358

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These recommendations, prepared by Committee 358, present a procedure for the design and analysis of reinforced and prestressed-concrete guideway structures for public transit. The document is specifically prepared to provide design guidance for elevated transit guideways. For items not covered in this document the engineer is referred to the appropriate highway and railway bridge design codes.

Limit states philosophy has been applied to develop the design criteria. A reliability approach was used in deriving load and resistance factors and in defining load combinations. A target reliability index of 4.0 and a service life of 75 years were taken as the basis for safety analysis. The reliability index is higher than the value generally used for highway bridges, in order to provide a lower probability of failure due to the higher consequences of failure of a guideway structure in a public transit system. The 75 year service life is comparable with that adopted by AASHTO for their updated highway bridge design specifications.

KEYWORDS: Box beams; concrete construction; cracking (fracturing); deformation; fatigue (materials); guideways; loads (forces); monorail systems: partial prestressing; precast concrete; prestressed concrete; prestress loss; rapid transit systems; reinforced concrete; serviceability; shear properties: structural analysis; structural design: T-beams; torsion; vibration.

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1. Scope

These recommendations are intended to provide public agencies, consultants, and other interested personnel with comprehensive criteria for the design and analysis of concrete guideways for public transit systems. They differ from those given for bridge design in ACI 343R, AASHTO bridge specifications, and the AREA manual of standard practice.

The design criteria specifically recognize the unique features of concrete transit guideways, namely, guideway/vehicle interaction, rail/structure interaction, special fatigue requirements, and esthetic requirements in urban areas. The criteria are based on current state-of-the-art practice for moderate-speed [up to 100 mph (160 km/h)] vehicles. The application of these criteria for advanced technologies other than those discussed in this report, require an independent assessment.

ACI 343R is referenced for specific items not covered in these recommendations. These references include materials, construction considerations, and segmental construction.

1.2-Definitions

The following terms are defined for general use in this document. For a comprehensive list of terms generally used in the design and analysis of concrete structures, the reader is referred to Chapter 2 of ACI 318 and to ACI 116R. The terminology used in this document conforms with these references.

**Broken rail** - The fracture of a continuously welded rail.

**Concrete** - A mixture of portland cement or any other hydraulic cement, fine aggregate, coarse aggregate, and water, with or without admixtures.

**Continuously welded rail** - Running rails that act as a continuous structural element as a result of full penetration welding of individual lengths of rail; continuously welded rails may be directly fastened to the guideway, in which case their combined load effects must be included in the design.

**Dead load** - The dead weight supported by a member, as defined in Chapter 3, without load factors.

**Design load** - All applicable loads and forces and their load effects such as, moments and shears used to proportion members; for design according to Chapter 5, design load refers to load without load factors; for design according to Chapter 6, design load refers to loads multiplied by appropriate load factors, as given in Chapter 4.

**Flexural natural frequency** - The first vertical frequency of vibration of an unloaded guideway, based on the flexural stiffness and mass distribution of the superstructure.

**Live load** - The specified live load, without load factors.

**Load factor** - A factor by which the service load is multiplied to obtain the design load.

**Service load** - The specified live and dead loads, without load factors.

**Standard vehicle** - The maximum weight of the vehicle used for design; the standard vehicle weight should allow for the maximum number of seated and standing passengers and should allow for any projected vehicle weight increases if larger vehicles or trains are contemplated for future use.

1.3 - Notation

\[ a = \text{center-to-center distance of shorter dimension of closed rectangular stirrups, in. (mm). Section 5.5.3} \]

\[ a_1 = \text{side dimension of a square post-tensioning anchor, or lesser dimension of a rectangular post-tensioning anchor, or side dimension of a square equivalent in area to a circular post-tensioning anchor, in. (mm). Section 5.8.2.1} \]

\[ a_2 = \text{minimum distance between the center-lines} \]
of anchors, or twice the distance from the centerline of the anchor to the nearest edge of concrete, whichever is less, in. (mm). Section 5.8.2.1
\[ A = \text{effective tension area of concrete surrounding the main tension reinforcing bars and having the same centroid as that reinforcement, divided by the number of bars, in.}^2 \text{ (mm}^2\text{);} \text{ when the main reinforcement consists of several bar sizes,}\]

\[ \text{the number of bars should be computed as the total steel area divided by the area of the largest bar used. Section 5.8.1}\]
\[ A = \text{exposed area of a pier perpendicular to the direction of stream flow, ft}^2 \text{ (m}^2\text{). Section 3.3.4}\]
\[ A_{bs} = \text{area of nonprestressed reinforcement located perpendicular to a potential bursting crack, in.}^2 \text{ (mm}^2\text{). Section 5.8.2.1}\]
\[ A_{oh} = \text{Area enclosed by the centerline of closed transverse torsion reinforcement, in.}^2 \text{ (mm}^2\text{). Section 5.5.3}\]
\[ A_r = \text{Cross-sectional area of a rail, in.}^2 \text{ (mm}^2\text{).}\]
\[ A_s' = \text{Area of compression reinforcement, in.}^2 \text{ (mm}^2\text{).}\]
\[ A_t = \text{Area of one leg of a closed stirrup resisting torsion within a distance, in.}^2 \text{ (mm}^2\text{).}\]
\[ A_v = \text{Area of shear reinforcement within a distance, or area of shear reinforcement perpendicular to main reinforcement within a distance for deep beams, in.}^2 \text{ (mm}^2\text{).}\]
\[ b = \text{Width of compressive face of member, in. (mm).}\]
\[ = \text{Center-to-center distance of longer dimension of closed rectangular stirrup, in. (mm). Section 5.5.3}\]
\[ b_k = \text{Width of concrete in the plane of a potential bursting crack, in. (mm). Section 5.8.2}\]
\[ BR = \text{Broken rail forces.}\]
\[ C_d = \text{Horizontal wind drag coefficient.}\]
\[ C_D = \text{Flowing water drag coefficient.}\]
\[ C_e = \text{Wind exposure coefficient.}\]
\[ C_f = \text{Wind gust effect coefficient.}\]
\[ C_{fI} = \text{Centrifugal force, kip (kN).}\]
\[ CL = \text{Collision load, kip (kN).}\]
\[ CR = \text{Forces due to creep in concrete, kip (kN).}\]
\[ d = \text{Distance from extreme compressive fiber to centroid of tension reinforcement, in. (mm).}\]
\[ d_c = \text{Thickness of concrete cover measured from the extreme tensile fiber to the center of the bar located closest thereto, in. (mm).}\]
\[ D = \text{Dead load.}\]
\[ DR = \text{Transit vehicle mishap load, due to vehicle derailment, kip (kN).}\]
\[ e = \text{Base of Napierian logarithms.}\]
\[ E_c = \text{Modulus of elasticity of concrete, psi (Pa).}\]
\( g \) = Acceleration due to gravity = 32.2 ft/sec\(^2\) (9.807 m/sec\(^2\)).

\( h \) = Overall thickness of member, in. (mm).

\( h_f \) = Compression flange thickness of I-and T-sections, in. (mm).

\( H \) = Ambient relative humidity. Section 3.4.4

\( H \) = Height from ground level to the top of the superstructure. Section 3.3.2

\( HF \) = Hunting force.

\( I \) = Impact factor.

\( ICE \) = Ice pressure.

\( I_{cr} \) = Moment of inertia of cracked section transformed to concrete, in.\(^4\) (m\(^4\)).

\( I_e \) = Effective moment of inertia for computation of deflections, neglecting the reinforcement, in.\(^4\) (m\(^4\)). Chapter 5

\( I_g \) = Moment of inertia of the gross concrete section about its centroidal axis neglecting reinforcement, in.\(^4\) (m\(^4\)).

\( jd \) = Distance between tensile and compression forces at a section based on an elastic analysis, in. (mm).

\( k_r \) = Average creep ratio.

\( k_t \) = \( k_r \), as a function of time \( t \).

\( k_v \) = A function of \( r_v \) for creep and shrinkage strains.

\( L \) = Span length, ft (m).

\( L_F \) = Live load.

\( L_{Fe} \) = Emergency longitudinal braking force.

\( L_{Fn} \) = Normal longitudinal braking force.

\( M \) = Mass per unit length, lb/in.-sec\(^2\) (kg/m).

\( M_n \) = Maximum moment in member at stage for which deflection is being computed, lb-in. (N-mm).

\( M_{cr} \) = Cracking moment, lb-m (N-mm).

\( P^S \) = Forces and effects due to prestressing.

\( q \) = Dynamic wind pressure, psf (MPa).

\( r_v \) = Volume-to-surface-area ratio, (volume per unit length of a concrete section divided by the area in contact with freely moving air), in. (mm).

\( r/h \) = Ratio of base radius to height of transverse deformations of reinforcing bars; when actual value is not known, use 0.3.

\( R \) = Radius of curvature, ft (m). Chapter 3

\( s \) = Shear or torsion reinforcement spacing in a direction parallel to the longitudinal reinforcement, in. (mm).

\( s \) = Spacing of reinforcement, in. (mm), Section 5.8.2

\( S \) = Service load combinations. Chapters 4 and 5.

\( SF \) = Stream flow load, lb (N). Chapter 3.

\( SH \) = Forces due to shrinkage in concrete.

\( t \) = Time, days.

\( T \) = Loads due to temperature or thermal gradient in the structure exclusive of rail forces. Chapter 4.

\( T \) = Time-dependent factor for sustained load. Section 5.7.2

\( \Delta T \) = Change in torsion at section due to fatigue loadings. Section 5.5.3

\( T_0 \) = Stress-free temperature of rail.

\( T_f \) = Final temperature in the continuously welded rail.

\( U \) = Ultimate load combinations.

\( \Delta V \) = Change in shear at section due to fatigue loadings, kip (kN). Section 5.5.3.

\( V \) = Velocity of water, wind, or vehicle, ft/sec (m/sec). Chapter 3.

\( V_{CF} \) = Vehicle crossing frequency, Hz. Section 3.3.1.

\( w_c \) = Unit weight of concrete, lb/ft\(^3\) (kg/m\(^3\)).

\( W \) = Wind load. Chapter 3.

\( W/L \) = Wind load on live load. Chapters 3 and 4.

\( W_S \) = Wind load on structure. Chapters 3 and 4.

\( x_m \) = Location of maximum bursting stress, measured from the loaded face of the end block, in. (mm).

\( y_t \) = Distance from the centroidal axis of cross section, neglecting the reinforcement, to the extreme fiber in tension, in. (mm).

\( z \) = A quantity limiting distribution of flexural reinforcement.

\( \alpha \) = Coefficient of thermal expansion. Chapter 3.

\( \gamma \) = Mass density of water, lb/ft\(^3\) (kg/m\(^3\)).

\( \varepsilon_i \) = Initial elastic strain.

\( \varepsilon_{cr} \) = Concrete creep strain at time \( t \).

\( \varepsilon_{sh} \) = Concrete shrinkage strain at time \( t \).

\( \varepsilon_{sh} \) = Concrete shrinkage strain at \( t = \infty \).

\( \omega \) = Angle in degrees between the wind force and a line normal to the guideway centerline.

\( \chi \) = Multiplier for additional long-time deflection as defined in Section 5.7.2.

\( \rho \) = Density of air in Section 3.3.2

\( \rho_{br} \) = Ratio of nonprestressed reinforcement located perpendicular to a potential bursting crack in Section 5.8.2.

\( \rho' \) = Compression reinforcement ratio = \( A_c / A_d \).

\( \phi \) = Strength reduction factor.

\( \psi \) = A parameter used to evaluate end block stresses. Section 5.8.2.1.

### 1.4- SI Equivalents

The equations contained in the following chapters are all written in the U.S. inch-pound system of measurements. In most cases, the equivalent SI (metric) equation is also given; however, some equations do not have definitive SI
equivalents. The reader is referred to ACI 318M for a consistent metric or SI presentation. In either case, the engineer must verify that the units are consistent in a particular equation.

1.5-Abbreviations
The following abbreviations are used in this report:

AASHTO American Association of State Highway and Transportation Officials
ACI American Concrete Institute
AREA American Railway Engineering Association
ASTM American Society for Testing and Materials
AWS American Welding Society
CRSI Concrete Reinforcing Steel Institute
FRA Federal Railway Administration, U.S. Department of Transportation

CHAPTER 2 - GENERAL DESIGN CONSIDERATIONS

2.1 - Scope
2.1.1 - General
Transit structures carry frequent loads through urban areas. Demands for esthetics, performance, cost, efficiency and minimum urban disruption during construction and operation are greater than for most bridge structures. The design of transit structures requires an understanding of transit technology, constraints and impacts in an urban environment, the operation of the transit system and the structural options available.

The guideway becomes a permanent feature of the urban scene. Therefore, materials and features should be efficiently utilized and built into the guideway to produce a structure which will support an operating transit system as well as fit the environment.

These guidelines provide an overview of the key issues to be considered in guideway design. They are intended to be a minimum set of requirements for materials, workmanship, technical features, design, and construction which will produce a guideway that will perform satisfactorily. Serviceability and strength considerations are given in this report. Sound engineering judgment must be used in implementing these recommendations.

2.1.2 - Guideway Structures
The guideway structure must support the transit vehicle, guide it through the alignment and restrain stray vehicles. Guidance of transit vehicles includes the ability to switch vehicles between guideways. The guideway must generally satisfy additional requirements, such as providing emergency evacuation, supporting wayside power distribution services and housing automatic train control cables.

Within a modern transit guideway, there is a high degree of repeatability and nearly an equal mix of tangent and curved alignment. Guideways often consist of post-tensioned concrete members. Post-tensioning may provide principal reinforcement for simple-span structures and continuity reinforcement for continuous structures. Bonded post-tensioned tendons are recommended for all primary load-carrying applications and their use is assumed in this report. However, unbonded tendons may be used where approved, especially for strengthening or expanding existing structures.

2.13 - Vehicles
Transit vehicles have a wide variety of physical configurations, propulsion, and suspension systems. The most common transit vehicles are steel-wheeled vehicles running on steel rails, powered by conventional guidance systems. Transit vehicles also include rubber-tired vehicles, and vehicles with more advanced suspension or guidance systems, such as air-cushioned or magnetically levitated vehicles. Transit vehicles may be configured as individual units or combined into trains.

2.2 - Structural Considerations
2.2.1 - General
Transit systems are constructed in four types of right-of-way: exclusive, shared-use rail corridor, shared-use highway corridor, and urban arterial. The constraints of the right-of-way affect the type of structural system which can be deployed for a particular transit operation. Constraints resulting from the type of right-of-way may include limited construction access, restricted working hours, limits on environmental factors such as noise, dust, foundation and structure placement, and availability of skilled labor and equipment.

Three types of concrete girders are used for transit superstructures. Namely, precast, cast-in-place, and composite girders. The types of guideway employed by various transit systems are listed in the Committee 358 State-of-the-Art Report on Concrete Guideways.

2.2.2 - Precast Girder Construction
When site conditions are suitable, entire beam elements are prefabricated and transported to the site. Frequently, box girder sections are used for their torsional stiffness, especially for short-radius curves. Some transit systems having long-radius
horizontal curves have used double-tee beams for the structure.

Continuous structures are frequently used. Precast beams are made continuous by developing continuity at the supports. A continuous structure has less depth than a simple-span structure and increased structural redundancy. Rail systems using continuously welded rail are typically limited to simple-span or two-span continuous structures to accommodate thermal movements between the rails and the structure. Longer lengths of continuous construction are used more readily in systems with rubber tired vehicles.

Segmental construction techniques may be used for major structures, such as river crossings or where schedule or access to the site favors delivery of segmental units. The use of segmental construction is discussed in ACI 343R.

2.2.3 - Cast-in-place Structures
Cast-in-place construction is used when site limitations preclude delivery of large precast elements. Cast-in-place construction has not been used extensively in modern transit structures.

2.2.4 - Composite Structures
Transit structures can be constructed in a similar manner to highway bridges, using precast concrete or steel girders with a cast-in-place composite concrete deck. Composite construction is especially common for special structures, such as switches, turnouts and long spans where the weight of an individual precast element limits its shipping to the site. The girder provides a working surface which allows accurate placement of transit hardware on the cast-in-place deck.

2.3- Functional Considerations
2.3.1- General
The functions of the structure are to support present and future transit applications, satisfy serviceability requirements, and provide for safety of passengers. The transit structure may also be designed to support other loads, such as automotive or pedestrian traffic. Mixed use applications are not included in the loading requirements of Chapters 3 and 4.

2.3.2 - Safety Considerations
Considerations for a transit structure must include transit technology, human safety and external safety, in accordance with the requirements of NFPA 130, "Fixed Guideway Transit Systems.

Transit technology considerations include both normal and extreme longitudinal, lateral, and vertical loads of the vehicle, as well as passing clearances for normal and disabled vehicles, vehicle speeds, environmental factors, transit operations, collision conditions, and vehicle retention.

Human safety addresses emergency evacuation and access, structural maintenance, fire control and other related subjects. Transit operations require facilities for evacuating passengers from stalled or disabled vehicles. These facilities should also enable emergency personnel to access such vehicles. In most cases, emergency evacuation is accomplished by a walkway, which may be adjacent to the guideway or incorporated into the guideway structure. The exact details of the emergency access and evacuation methods on the guideway should be resolved among the transit operator, the transit vehicle supplier, and the engineer. The National Fire Protection Association (NFPA) Code, Particularly NFPA - 130, gives detailed requirements for safety provisions on fixed guideway transit systems.

External safety considerations include safety precautions during construction, prevention of local street traffic collision with the transit structure, and avoidance of navigational hazards when transit structures pass over navigable waterways.

2.3.3-Lighting
The requirements for lighting of transit structures should be in accordance with the provisions of the authority having jurisdiction. Such provisions may require that lighting be provided for emergency use only, or for properties adjacent to the guideway structure, or, alternatively, be deleted altogether.

2.3.4-Drainage
To prevent accumulation of water within the track area, transit structures should be designed so that surface runoff is drained to either the edge or the center of the superstructure, whereupon the water is carried longitudinally.

Longitudinal drainage of transit structures is usually accomplished by providing a longitudinal slope to the structure; a minimum slope of 0.5 percent is preferred. Scuppers or inlets, of a size and number that adequately drain the structure should be provided. Downspouts, where required, should be of a rigid, corrosion-resistant material not less than 4 in. (100 mm) and preferably 6 in. (150 mm) in the least dimension; they should be provided with cleanouts. The details of the downspout and its deck inlet and outlet should be such as to prevent the discharge of water against any portion of the structure and should prevent erosion at ground level. Slopes should be arranged so that run-off drains away from stations. Longitudinal grades to assure drainage should be
coordinated with the natural topography of the site to avoid an unusual appearance of the structure.

Architectural treatment of exposed downspouts is important. When such treatment becomes complicated, the use of internal or embedded downspouts becomes preferable. For internal or external downspouts, consideration must be given to the prevention of ice accumulation in cold-weather climates. This may require localized heating of the drain area and the downspout itself. All overhanging portions of the concrete deck should be provided with a drip bead or notch.

2.3.5 - Expansion Joints and Bearings

Expansion joints should be provided at span ends; this allows the beam ends to accommodate movements due to volumetric changes in the structure. Joints should be designed to reduce noise transmission and to prevent moisture from seeping to the bearings. Adequate detailing should be provided to facilitate maintenance of bearings and their replacement, when needed, during the life of the structure.

Aprons or finger plates, when used, should be designed to span the joint and to prevent the accumulation of debris on the bearing seats. When a waterproof membrane is used, the detail should be such that penetration of water into the expansion joint and the bearing seat is prevented.

2.3.6 - Durability

In order to satisfy the design life of 75 years or more, details affecting the durability of the structure should be given adequate consideration; these should include materials selection, structural detailing, and construction quality control.

Materials selection includes the ingredients of concrete and its mix design, allowing for a low water-cement ratio and air entrainment in areas subject to freeze-thaw action. Epoxy-coated reinforcement and chloride-inhibitor sealers may be beneficial if chloride use is anticipated as part of the winter snow-clearing operations or if the guideway may be exposed to chloride-laden spray from a coastal environment or to adjacent highways treated with deicing chemicals.

In structural detailing, both the reinforcement placement and methods to prevent deleterious conditions from occurring should be considered. Reinforcement should be distributed in the section so as to control crack distribution and size. The cover should provide adequate protection to the reinforcement.

Incidental and accidental loadings should be accounted for and adequate reinforcement should be provided to intersect potential cracks. Stray currents, which could precipitate galvanic corrosion, should be accounted for in the design of electrical hardware and appurtenances and their grounding.

Construction quality control is essential to ensure that the design intent and the durability considerations are properly implemented. Such quality-control should follow a pre-established formal plan with inspections performed as specified in the contract documents.

To satisfy a 75-year service life, regular inspection and maintenance programs to ensure integrity of structural components should be instituted. These programs may include periodic placement of coatings, sealers or chemical neutralizers.

2.4 - Economic Considerations

The economy of a concrete guideway is measured by the annual maintenance cost and capitalized cost for its service life. It is particularly important that the design process give consideration to the cost of operations and maintenance and minimize them. Therefore, consideration must be given to the full service life cost of the guideway structure. The owners should provide direction for the establishment of cost analyses. Economy is considered by comparative studies of reinforced, prestressed, and partially prestressed-concrete construction. Trade-offs should be considered for using higher grade materials for sensitive areas during the initial construction against the impact of system disruption at a later date if the transit system must be upgraded. For example, higher quality aggregates may be selected for the traction surface where local aggregates have a tendency to polish with continuous wear.

2.5 - Urban Impact

2.5.1 - General

The guideway affects an urban environment in three general areas: visual impact, physical impact, and access of public safety equipment. Visual impact includes both the appearance of the guideway from surrounding area and the appearance of the surrounding area from the guideway. Physical impacts include placement of columns and beams and the dissipation of noise, vibration, and electromagnetic radiation. Electromagnetic radiation is usually a specific design consideration of the vehicle supplier. Public safety requires provision for fire, police, and emergency service access and emergency evacuation of passengers.

2.5.2 - Physical Appearance

A guideway constructed in any built-up environment should meet high standards of esthetics for physical appearance. The size and configuration of the guideway elements should en-
sure compatibility with its surroundings. While the range of sizes and shapes is unlimited in the selection of guideway components the following should be considered:

a. View disruption  
b. Shade and shelter created by the guideway  
c. Blockage of pedestrian ways  
d. Blockage of streets and the effect on traffic and parking  
e. Impairment of sight distances for traffic below  
f. Guideway mass as it relates to adjacent structures  
g. Construction in an urban environment  
h. Methods of delivery of prefabricated components and cast-in-place construction  
i. Interaction with roadway and transit vehicles  
j. Visual continuity

Attention to final detailing is important. Items to be considered should include:

a. Surface finish  
b. Color  
c. Joint detailing  
d. Provision to alleviate damage from water dripping from the structure  
e. Control and dissipation of surface water runoff  
f. Differences in texture and color between cast-in-place and precast elements

2.5.3 -Sightliness

In the design of a guideway the view of the surroundings from the transit system itself should be considered. The engineer should be aware that patrons riding on the transit system will have a view of the surroundings which is quite different from that seen by pedestrians at street level. As such, the guideway placement and sightliness should reflect a sensitivity to intrusion on private properties and adjacent buildings. In some cases, the use of noise barriers and dust screens should be considered.

The view of the guideway from a higher vantage point has some importance. The interior of the guideway should present a clean, orderly appearance to transit patrons and adjacent observers. Any supplemental cost associated with obtaining an acceptable view must be evaluated.

2.5.4 -Noise Suppression

A transit system will add to the ambient background noise. Specifications for new construction generally require that the wayside noise 50 ft. (15 m) from the guideway not exceed a range of 65 to 75 dBA. This noise is generated from on-board vehicle equipment such as propulsion and air-conditioning units, as well as from vehicle/track interaction, especially when jointed rail is used.

It is normally the responsibility of the vehicle designer to control noise emanating from the vehicle. Parapets and other hardware on the guideway structure should be designed to meet general or specific noise suppression criteria. Determination of these criteria is made on a case-by-case basis, frequently in conjunction with the vehicle supplier.

2.5.5- Vibration

Transit vehicles on a guideway generate vibrations which may be transmitted to adjacent structures. For most rubber tired transit systems, this groundborne vibration is negligible. In many rail transit systems, especially those systems with jointed rails, the noise and the vibration can be highly perceptible. In these situations, vibration isolation of the structure is necessary.

2.5.6 -Emergency Services Access

A key concern in an urban area is the accessibility to buildings adjacent to a guideway by fire or other emergency equipment. Within the confined right-of-way of an urban street, space limitations make this a particularly sensitive concern. In most cases a clearance of about 15 ft. (5 m) between the face of a structure and a guideway provides adequate access. Access over the top of a guideway may not represent a safe option.

2.6- Transit Operations

2.6.1 - General

Once a transit system is opened for service, the public depends on its availability and reliability. Shutdowns to permit maintenance, operation, or expansion of the system can affect the availability and reliability of the transit system. These concerns often lead to long-term economic, operational, and planning analyses of the design and construction of the transit system.

In most transit operations, a shutdown period between the hours of 1:00 a.m. and 5:00 a.m. (0100 and 0500) can be tolerated; slightly longer shutdowns are possible in certain locations and on holidays. It is during this shutdown period that routine maintenance work is performed.

Many transit systems also perform maintenance during normal operating hours. This practice tends to compromise work productivity and guideway access rules and operations in order to provide a safe working space. The transit operators should provide the engineer with guidelines regarding capital cost objectives and their operation and maintenance plans.

2.6.2 -Special Vehicles
Transit systems frequently employ special vehicles for special tasks, such as, retrieving disabled vehicles and repairing support or steering surfaces. While the design may not be predicated on the use of special vehicles, their frequency of use, weights, and sizes must be considered in the design.

2.6.3 - Expansion of System

Expansion of a transit system can result in substantial disruption and delay to the transit operation while equipment, such as switches, are being installed. In the initial design and layout of a transit system, consideration should be given to future expansion possibilities. When expansion is contemplated within the foreseeable future after construction and the probable expansion points are known, provisions should be incorporated in the initial design and construction phases.

2.7- Structure/Vehicle Interaction

2.7.1- General

Vehicle interaction with the guideway can affect its performance as related to support, steering, power distribution and traction components of the system. It is usually considered in design through specification of serviceability requirements for the structure. In the final design stage close coordination with the vehicle supplier is imperative.

2.7.2- Ride Quality

2.7.2.1- General

Ride quality is influenced to a great degree by the quality of the guideway surface. System specifications usually present ride quality criteria as lateral, vertical and longitudinal accelerations and jerk rates (change in rate of acceleration) as measured inside the vehicle. These specifications must be translated into physical dimensions and surface qualities on the guideway and in the suspension of the vehicle. The two elements that most immediately affect transit vehicle performance are the support surface and steering surface.

2.7.2.2 - Support Surface

The support surface is basically the horizontal surface of the guideway which supports the transit vehicle against the forces of gravity. It influences the vehicle performance by the introduction of random deviations from a theoretically perfect alignment. These deviations are input to the vehicle suspension system. The influence of the support surface on the vehicle is a function of the type of the suspension system, the support medium (e.g., steel wheels or rubber tire), and the speed of the vehicle.

There are three general components of support surfaces which must be considered. Namely, local roughness, misalignment, and camber. Local roughness is the amount of distortion on the surface from a theoretically true surface. In most transit applications, the criterion of a l/8-inch (3 mm) maximum deviation from a 10 ft. (3 m) straightedge, as given in ACI 117, is used.

With steel rails, a Federal Railway Administration (FRA) Class 62 tolerance is acceptable. The FRA provision include provisions for longitudinal and transverse (roll) tolerances. These tolerances are consistent with operating speeds of up to 50 mph (80 km/h). Above these speeds, stricter tolerance requirements have to be applied.

Vertical misalignment most often occurs when adjacent beam ends meet at a column or other connection. There are two types of misalignment which must be considered. The first, is a physical displacement of adjacent surfaces. This occurs when one beam is installed slightly lower or higher than the adjacent beam. These types of misalignment should be limited to l/16 in. (1.5 mm) as specified by ACI 117.

The second type of vertical misalignment occurs when there is angular displacement between beams. Such an angular displacement may result from excessive deflection, sag, or camber. Excessive camber or sag creates a discontinuity which imparts a noticeable input to the vehicle suspension system.

In the design and construction of the beams the effects of service load deflection, initial camber and long-time deflections should be considered. There is no clear definition on the amount of angular discontinuity that can be tolerated at a beam joint. However, designs which tend to minimize angular discontinuity generally provide a superior ride. Continuous guideways are particularly beneficial in controlling such misalignment.

Camber or sag in the beam can also affect ride quality. Consistent upward camber in structures with similar span lengths can create a harmonic vibration in the vehicle resulting in a dynamic amplification, especially in continuous structures. When there are no specific deflection or camber criteria cited for a project, the designer should account for these dynamic effects by analytical or simulation techniques. The deflection compatibility requirements between structural elements and station platform edges should be accounted for.

2.7.2.3- Steering Surface

The steering surface provides a horizontal input to the vehicle. The steering surfaces may be either the running rails for a flanged steel-wheel-rail system or the concrete or steel vertical surfaces that are integrated into the guideway struc-
NORMAL CONFIGURATION
STEERING WHEELS
CENTERED IN THE GUIDEWAY

ROLLED CONFIGURATION
RIGHT STEERING WHEEL
COMPRESSED AGAINST
THE GUIDEWAY GENERATING A
SPURIOUS STEERING INPUT.

Fig. 2.7.2.3- Interaction between support and steering

ture, for a rubber tired system. The condition of the steering surface is particularly important since few vehicles have sophisticated lateral suspension systems. In most existing guideways, the tolerance of a 1/8 in. (3 mm) deviation from a 10 ft. (3 m) straightedge, specified by ACI-117, corrected for horizontal curvature, has proven to be adequate for rubber tired vehicles operating at 35 mph (56 km/h) or less. In steel-rail systems, an FRA Class 6.2 rail tolerance has generally proven to be satisfactory for speeds up to 70 mph (112 km/h). Other tolerance limits are given in Table 2.7.2.3.

There is a particular interaction between the steering surface and the support surface, which is technology dependent and requires specific consideration by the engineer. This interaction results from a coupling effect which occurs when a vehicle rolls on the primary suspension system, causing the steering mechanism to move up and down (Fig. 2.7.2.3). The degree of this up and down movement is dependent on the steering mechanism which is typically an integral part of the vehicle truck (bogie) system, and the stiffness of the primary suspension which is also within the truck assembly.

Depending upon the relationship between the support and the steering surfaces, and the support and guidance mechanisms of the vehicle (primary, in the case of rubber tired system) a couple can be created between the two, which causes a spurious steering input into the vehicle. There are no general specifications for this condition. The engineer should be aware that this condition can exist and, if there is a significant distance separating the horizontal and vertical contact surfaces, additional tolerance requirements for the finished surfaces have to be imposed. This is in order to reduce the considerable steering input, which can cause over or under steering, which leads to an accelerated wear of components and degraded ride comfort.

<table>
<thead>
<tr>
<th>Table 2.7.2.3 Track Construction Tolerances</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type and Class of Track</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Direct Fixation and Ballasted Main Line</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Ballasted Yard and Secondary Lines</td>
</tr>
<tr>
<td></td>
</tr>
</tbody>
</table>

Notes: - Tolerances are all positive, unless otherwise indicated.
- Dimensions are in inches (mm)
- H=Horizontal, V=Vertical, Sup.=Superelevation
- Total Deviation is measured between the theoretical and the actual alignments at any point along the track.
- Variations from theoretical gage, cross level and superelevation are not to exceed 1/8 in. (3 mm) per 15'-6 (4.7 m) of track.
- The total Deviation in platform areas should be zero towards the platform and 1/4 in (6 mm) away from the platform.
2.7.3 - Traction Surfaces

Transit vehicles derive their traction from the physical contact of the wheels with the concrete or running rail or through an electromagnetic force. In those systems where traction occurs through physical contact with the guideway, specific attention must be given to the traction surface.

In automated transit, the traction between the wheel and the reaction surface is essential to ensure a consistent acceleration and a safe stopping distance between vehicles. It is also important for automatic control functions. The engineer should determine the minimum traction required for the specific technology being employed. If the traction surface is concrete, appropriate aggregates should be provided in the mix design to maintain minimum traction for the working life of the structure.

Operation in freezing rain or snow may also affect traction on the guideway. The engineer should determine the degree of traction maintenance required under all operating conditions. If full maintenance is required, then the engineer should examine methods to mitigate the effects of snow or freezing rain. These mitigating effects may include heating the guideway, enclosing the guideway, or both.

If deicing chemicals are contemplated, proper material selection and protection must be considered. Corrosion protection may require consideration of additional concrete cover, sealants, epoxy-coated reinforcing steel, and special concrete mixes.

2.7.4 - Electrical Power Distribution

There are two components to electrical power distribution: the wayside transmission of power to the vehicle and the primary power distribution to the guideway. The wayside power distribution to the vehicle is normally done through power rails or through an overhead catenary. Provision must be made on the guideway for the mounting of support equipment for the installation of this wayside power.

For systems using steel running rails, where the running rail is used for return current, provisions must also be made to control any stray electrical currents which may cause corrosion in the guideway reinforcement or generate other stray currents in adjacent structures or utilities.

The primary power distribution network associated with a guideway may require several substations along the transit route. Power must be transmitted to the power rails on the guideway structure at various intervals. This is usually done through conduits mounted on or embedded in the guideway structure.

Internal conduits are an acceptable means of transmitting power; they may be used to route power from the substation to the guideway. However, access to internal conduits is difficult to detail and construct. Sufficient space must be provided within the column-beam connection and within the beam section for the conduit turns; space must also be provided for safe electrical connections. Exterior conduits can detract from the guideway appearance and can cause increased maintenance requirements.

2.7.5 - Special Equipment

A guideway normally carries several pieces of special transit equipment. This equipment may consist of switches, signaling, command and control wiring, or supplemental traction and power devices. The specialty transit supplier should provide the engineer with explicit specifications of special equipments and their spatial restrictions. For example, the placement of signaling cables within a certain distance of the wayside power rails or reinforcing steel may be restricted.

The transit supplier should also provide the engineer with the forces and fatigue requirements of any special equipment so that proper connections to the structure can be designed and installed. An example of connection requirements would be linear induction motor reaction rail attachments.

When no system supplier has been selected, the engineer must provide for the anticipated services and equipment. In this instance, a survey of the needs of potential suppliers for the specific application may be required prior to design.

2.8 - Geometries

2.8.1 - General

The geometric alignment of the transit line can have a substantial impact on the cost of the system. Standardization of the guideway components can lead to cost savings. During the planning and design stages of the transit system, the benefits of standardizing the structural elements, in terms of ease and time of construction and maintenance, should be examined and the effective options implemented.

2.8.2 - Standardization

Straight guideway can be produced at a lower cost than curved guideway. Geometric alignments and column locations that yield a large number of straight beams tend to be cost-effective. Physical constraints at the ground influence column locations. However, when choices are available, the placement of columns to generate straight beams, as opposed to those with a slight horizontal or vertical curvature, will usually prove to be more
cost effective.

Standardization and coordination of the internal components and fixtures of the guideway also tends to reduce overall cost. These include inserts for power equipment, switches, or other support elements. Methods to achieve this are discussed in Section 2.9.3.

2.8.3 - Horizontal Geometry

The horizontal geometry of a guideway alignment consists of circular curves connected to tangent elements with spiral transitions. Most types of cubic spirals are satisfactory for the transition spiral. The vehicle manufacturer may provide additional constraints on the selection of a spiral geometry to match the dynamic characteristics of the vehicle.

2.8.4 - Vertical Geometry

The vertical geometry consists of tangent sections connected by parabolic curves. In most cases, the radius of curvature of the parabolic curves is sufficiently long that a transition between the tangent section and the parabolic section is not required.

2.8.5 - Superelevation

Superelevation is applied to horizontal curves in order to partially offset the effect of lateral acceleration on passengers. To accomplish the required superelevation, the running surface away from the curve center is raised increasingly relative to that closer to the curve center. This results in the outer rail or wheel track being raised while the inner rail or wheel track being kept at the profile elevation. The amount of superelevation is a function of the vehicle speed and the degree of curvature. It is usually limited to a maximum value of 10 percent.

2.9 - Construction Considerations

2.9.1 - General

Construction of the guideway in an urban environment has an impact on the residents, pedestrians, road traffic, and merchants along the route. Consideration should be given to the cost and length of disruption, in terms of street closure and construction details.

2.9.2 - Street Closures and Disruptions

The amount of time that streets are closed and neighborhoods are disrupted should be kept to a minimum. Coordination with the public should begin at the planning stage. The selection of precast or cast-in-place concrete components and methods of construction depend on the availability of construction time and on the ease of stockpiling equipment and finished products at the proximity of the site. Construction systems which allow for rapid placement of footings and columns and for reopening of the street prior to the installation of beams, may have an advantage in the maintenance of local traffic.

2.9.3 - Guideway Beam Construction

Guideway beams may be cast-in-place or precast. In order to ascertain the preferred construction technique, the following items need to be considered early in the design process: typical section and alignment, span composition (uniform or variable), structure types, span-depth ratios, and major site constraints.

Cast-in-place construction offers considerable design and construction flexibility, however, it also requires a greater amount of support equipment on the site. This equipment, especially shoring and falsework, has to remain in place while the concrete cures.

Precast concrete beam construction offers the potential for reduced construction time on site and allows better quality control and assurance. Advantages of precast concrete are best realized when the geometry and the production methods are standardized.

Two types of guideway beam standardization appear to offer substantial cost benefits. Namely, modular construction and adjustable form construction.

Modular construction utilizes a limited number of beam and column types to make up the guideway. Thus, like a model train set, these beams are interwoven to provide a complete transit guideway. Final placement of steering surfaces and other system hardware on the modular elements provides the precise geometry necessary for transit operation. Modules may be complete beams. Segmental construction also typifies this construction technique.

An adjustable form allows the fabrication of curved beams to precisely match the geometric requirements at the site. For alignments where a substantial amount of variation in geometry is dictated by the site, this solution provides a high degree of productivity at a reasonable cost.

2.9.4 - Shipping and Delivery

Prior to the completion of final design, the engineer should be aware of limitations which may be placed upon the delivery of large precast elements. Weight limitations imposed by local departments of transportation, as well as dimensional limitations on turnoff radii, width, and length of beam elements, may play an important role in the final guideway design. The deployment of large cranes and other construction equipment along the site is also a consideration.
2.9.5- Approval Considerations
These recommendations for transit guideways are intended to provide procedures based on the latest developments in serviceability and strength design. Other pertinent regulations issued by state, federal, and local agencies should be considered.
Specific consideration should be given to the following:
- Alternative designs
- Environmental impact statements
- Air, noise, and water pollution statutes
- Historic and park preservation requirements
- Permits
- Life-safety requirements
- Construction safety requirements

2.9.6 -Engineering Documents
The engineering documents should define the work clearly. The project drawings should show all dimensions of the finished structure in sufficient detail to facilitate the preparation of an accurate estimate of the quantities of materials and costs and to permit the full realization of the design.
The contract documents should define test and inspection methods, as well as the allowable procedures and tolerances to ensure good workmanship, quality control, and application of unit costs, when required in the contract. The contractor’s responsibilities should be clearly defined. Where new or innovative structures are employed, suggested construction procedures to clarify the engineer’s intent should also be provided. Computer graphics or integrated data bases can assist in this definition.

2.10- Rails and Trackwork

2.10.1- General
Guideways for transit systems which utilize vehicles with steel wheels operating on steel rails require particular design and construction considerations, which include, rail string assembly, use of continuous structures, and attachment of the rails to the structure.
Two options exist for assembling the rails: They may be jointed with bolted connections in standard 39 ft. (11.9 m) lengths, or welded into continuous strings. The rails may be fastened directly to the structure or installed on tie-and-ballast.

2.10.2- Jointed Rail
The traditional method of joining rail is by bolted connections. Sufficient longitudinal rail movement can develop in these connections to prevent the accumulation of the thermal stresses along the length of the rails.
The space between the rail ends presents a discontinuity to the vehicle support and steering systems. Vehicle wheels hitting this discontinuity cause progressive deterioration of the joints, generate loud noise, reduce ride comfort, and increase the dynamic forces on the structure.
Because of these limitations, most modern transit systems use continuously welded rail. However, jointed rail conditions will exist in switch areas, maintenance yards and other locations where physical discontinuities are required. However, even in these areas, discontinuities can be reduced greatly by the use of bonded rail joints.

2.10.3 -Continuously Welded Rail

2.10.3.1 -General
To improve the ride quality and decrease track maintenance, individual rails are welded into continuous strings. There is no theoretical limit to the length of continuously welded rail if a minimum restraint is provided. Minimum rail restraint consists of prevention of horizontal or vertical buckling of rails and anchorage at the end of a continuous rail to prevent excessive rail gaps from forming at low temperatures, if accidental breaks in the rail should occur.
Continuously welded rail (CWR) has become the standard of the transit industry over the past several decades. The use of CWR requires particular attention to several design details, which include, thermal forces in the rails, rail break gap and forces, welding of CWR, and fastening of CWR to the structure. The principal variables used in the evaluation of rail forces are rail size in terms of its cross-sectional area, the characteristics of the rail fastener, the stiffness of the structural elements, rail geometry, and operational environment, in terms of temperature range.
In cases where accumulation of the thermal effects would produce conditions too severe for the structure, slip joints can be used. Slip joints allow limited movement between rail strings. They generally cause additional noise and require increased maintenance. Their use therefore is not desirable. Location of rail anchors and rail expansion joints will affect the design of the structure.

2.10.3.2 -Thermal Forces
Changes in temperature of continuously welded rails will develop stresses in the rail and in the structure. Rails are typically installed at a design stress-free ambient temperature, to reduce the risk of rail buckling at high temperatures and rail breaks at low temperatures. Depending upon the method of attachment of the rails to the structure, the structure should be designed for:
- Horizontal forces resulting from a rail break
- Radial forces resulting from thermal changes in the rails on horizontal or vertical curves
- End anchorage forces

2.10.3.3 - Rail Breaks
Continuously welded rails will, on occasion, fail in tension. This situation occurs because of rail wear, low temperature, defects in the rail, defects in a welded joint, fatigue or some combination of these effects. The structure should be designed to accommodate horizontal thrust associated with the break.

2.10.3.4 - Rail Welding
Continuous welded rail is accomplished by either the them-rite welding process or the electric flash butt welding process. Proper weld procedures should ensure that:

- Adjacent rail heads are accurately aligned
- Rails are welded at the predetermined stress-free ambient temperature
- Rail joint is clean of debris
- The finished weld is free of intrusions
- Weld is allowed to cool prior to tightening the fasteners.

Ultrasonic or x-ray inspection of the welds at random locations is suggested.

2.10.4 - Rail Installation
2.10.4.1 - General
Rails are attached to either cross ties on ballast or directly to the guideway structure. The preference in recent years has become direct rail fixation as a means of improving ride quality, maintaining rail tolerances, reducing maintenance costs, and reducing structure size.

2.10.4.2 - Tie and Ballast
Tie and ballast construction is the conventional method of installing rails at grade and occasionally on elevated structures. Ties are used to align and anchor the rails. Ballast provides an intermediate cushion between the rails and the structure, stabilizes the tracks, and prevents thermal forces to be transmitted from the rails to the structure.

Ballast substantially increases the structure dead load. Tie-and-ballast installations make control of rail break gaps difficult since the ties are not directly fastened to the primary structure. Rail breaks can develop horizontal, vertical, and angular displacements of the rail relative to the structure.

2.10.4.3 - Direct Fixation
Direct fixation of the rail to the structure is accomplished by means of mechanical rail fastener. Elastomeric pads are incorporated in the fastener to provide the required vertical and horizontal flex and provisions for adjustment between adjacent fasteners and the structure. The elastomeric pads also assist in the reduction of noise, vibration, and impact.

Important design and construction considerations for the direct fixation fasteners include:

- Method of attachment to the structure
- Vertical stiffness
- Allowance for horizontal and vertical adjustment
- Ability to restrain the rail against rollover
- Longitudinal restraint

Direct fixation fasteners are one of the most important elements in the design of the trackwork. They are subjected to a high number of cyclic loads and there are thousands of fasteners in place in any one project. Progressive failure does not generally create catastrophic results, but leads to a substantial maintenance effort and possible operational disruptions.

No industry wide specifications exist for the definition or procurement of direct fixation fasteners. A thorough examination of the characteristics and past performance of available fasteners, and the characteristics of the proposed transit vehicle should be undertaken prior to fastener selection for any specific installation.

2.10.4.4 - Continuous Structure
Direct fixation of continuous rail to a continuous structure creates a strain discontinuity at each expansion joint in the structure. Fasteners must be designed to provide adequate slip at these joints while still being able to limit the rail-gap size in the event of a rail break. In climates with extreme ranges in temperature [-40 F to +90 F (-40 C to +30 C)], structural continuity is generally limited to 200 to 300 ft. (60 to 90 m) lengths. In more moderate climates, longer runs of continuous structure may be possible.

REFERENCES*


CHAPTER 3 - LOADS

3.1 - General
The engineer should investigate all special, unusual, and standard loadings that may occur in the guideway being designed. Special or unusual loads may include emergency, maintenance, or evacuation equipment or conditions. The following loads commonly occur and are considered when assessing load effects on elevated guideway structures.

a. Sustained loads
   - Dead load
   - Earth pressure
   - External restraint forces
   - Differential settlement effects
   - Buoyancy

b. Transient loads
   - Live load and its derivatives
   - Wind
   - Loads due to ice
   - Loads due to stream current

c. Loads due to volumetric changes
   - Temperature
   - Rail-structure interaction
   - Shrinkage
   - Creep

d. Exceptional loads
   - Earthquake
   - Derailment
   - Broken rail
   - Collision loads at street level

e. Construction Loads
   - Dead Loads
   - Live Loads

3.2 - Sustained loads

3.2.1 - Dead Loads, \(D\)
Four components of dead load are considered:

- Weight of factory-produced elements
- Weight of cast-in-place elements
- Weight of trackwork and appurtenances which includes running and power rails, second-pour plinths and fasteners, barrier walls, and noise-suppression panels
- Weight of other ancillary components

3.2.2 - Other Sustained Loads
Loads from differential settlement, earth pressure, effects of prestress forces (PS) or external structural restraints should be included in the design, as they occur. The beneficial effects of buoyancy may only be included when its existence is ensured. References 3.2 and 3.11 may be used as guides to evaluate the effects of these sustained loads.

3.3 - Transient Loads

3.3.1 - Live Load and its Derivatives

3.3.1.1 - Vertical Standard Vehicle Loads, \(L\)
The vertical live load should consist of the weight of one or more standard vehicles positioned to produce a maximum load effect in the element under consideration. The weight and configuration of the maintenance vehicle are to be considered in the design. The weight of passengers should be computed on the basis of 175 lb (780 N) each and should comprise those occupying all the seats (the seated ones) and those who are standing in the rest of the space that does not have seats (standees). The number of standees shall be based on one passenger per 1.5 ft.\(^2\) (0.14 m\(^2\)).

For torsion-sensitive structures, such as monorails, the possibility of passengers being crowded on one side of the vehicle should be considered in the design.

3.3.1.2 - Impact Factor, \(I\)
The minimum dynamic load allowance shown in Table 3.3.1.2 should be applied to the vertical vehicle loads, unless alternative values based on tests or dynamic analysis are approved.

Definition of terms in the Table follow:

- \(VCF\) = \(\frac{\text{vehicle speed, ft/sec (m/sec)}}{\text{span length, ft (m)}}\) \hspace{1cm} (3-1)
- \(f_1\) = first mode flexural (natural) frequency of the guideway where,

\[
f_1 = \frac{\pi}{2\ell^2} \sqrt{\frac{E I_f}{M}} \hspace{1cm} (3-2)
\]

where

- \(\ell\) = span length, center-to-center of supports, in. (m)
- \(M\) = mass per unit length of the guideway, which includes all the sustained loads the beam carries including its own mass, lb/in.-sec\(^2\)/in. (kg/m)
Table 3.3.1.2 Dynamic Load Allowance (Impact)

<table>
<thead>
<tr>
<th>Structure Types</th>
<th>Rubber-tired and Continuously Welded Rail</th>
<th>Jointed rail</th>
</tr>
</thead>
<tbody>
<tr>
<td>Simple-span structures,</td>
<td>$J = \frac{VCF}{f_1} - 0.1$</td>
<td>$\geq 0.10$</td>
</tr>
<tr>
<td>Continuous-span structures,</td>
<td>$J = \frac{VCF}{2f_1} - 0.1$</td>
<td>$\geq 0.10$</td>
</tr>
</tbody>
</table>

$E_c$ = modulus of elasticity of the guideway, psi (Pa)

$I_g$ = moment of inertia of uncracked section of the guideway, in.\(^4\) (m\(^4\))

$VCF$ = Vehicle Crossing Frequency, Hz

The dynamic load allowance should not be applied to footings and piles.

3.3.1.3 - Centrifugal Force, $CF$

The centrifugal force, $CF$, acting radially through the center of gravity of the vehicle at a curved track may be computed from,

$$CF = \frac{V^2}{R_g} L, \text{lb} (N) \quad (3-3)$$

where,

$R$ = radius of curvature, ft (m)

$g$ = acceleration due to gravity, 32.2 ft/sec/sec (9.82 m/s\(^2\))

$V$ = maximum operating speed of the vehicle, ft/sec (m/s) and,

$L$ = the standard vehicle load, kips (kN)

The load, $L$, should be applied simultaneously with other load combinations (Chapter 4) in order to produce the maximum force effect on the structure.

3.3.1.4 - Hunting Force, $HF$

The hunting (or “nosing”) force, $HF$, is caused by the lateral interaction of the vehicle and the guideway. It should be applied laterally on the guideway at the point of wheel-rail contact, as a fraction of the standard vehicle load, $L$, as follows:

<table>
<thead>
<tr>
<th>Bogie type</th>
<th>Hunting force</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nonsteerable</td>
<td>0.08$L$</td>
</tr>
<tr>
<td>Steerable</td>
<td>0.06$L$</td>
</tr>
</tbody>
</table>

When centrifugal and hunting forces can act simultaneously, only the larger force need be considered.

For rail and structure design, the hunting force would be applied laterally by a steel wheel to the top of the rail at the lead axle of a transit train. It need not be applied for rubber tired systems; typically, LIM propelled vehicles run on steel-wheel-and-rail, and hence require consideration of hunting effects.

3.3.1.5 - Longitudinal Force, $LF$

The longitudinal force acts simultaneously with the vertical live load of a standard vehicle on all wheels. It may be applied in either direction: forward in braking or deceleration or reverse in acceleration. The longitudinal force should be applied as follows:

$Emergency braking, LF_e = 0.30L$

$Normal braking, \quad LF_n = 0.15L$

Continuously welded rail trackwork can distribute longitudinal forces to adjacent components of guideway structures. This distribution may be considered in design. Use of slip joints may prevent transfer and distribution of longitudinal forces.

3.3.1.6 - Service Walkway Loads

Live load on service or emergency walkways shall be based on 85 psf (4.0 kPa) of area. This load should be used together with empty vehicles on the guideway, since the walkway load is the result of vehicles being evacuated.
3.3.1.7-Loads on Safety Railing
The lateral load from pedestrian traffic on railings should be 100 lb/ft (1.5 kN/m) applied at the top rail.

3.3.2-Wind Loads, W

3.3.2.1-General
This section provides design wind loads for elevated guideways and special structures. Wind loads, based on the reference wind pressure, shall be treated as equivalent static loads as defined in Section 3.5.3.

Wind forces are applied to the structure and to the vehicles in accordance with the load combinations in Chapter 4. WL is used to designate wind loads applied to vehicle, while WS indicates wind loads applied to the structure only.

The net exposed area is defined as the net area of a body, member, or combination of members as seen in elevation. For a straight superstructure, the exposed frontal area is the sum of the areas of all members, including the railings and deck systems, as seen in elevation at 90 degrees to the longitudinal axis. For a structure curved in plan, the exposed frontal area is taken normal to the beam centerline and is computed in a similar manner to tangent structures.

The exposed plan area is defined as the net area of an element as seen in plan from above or below. In the case of a superstructure, the exposed plan area is the plan area of the deck and that of any laterally protruding railings, members or attachments.

The gust effect coefficient is defined as the ratio of the peak wind-induced response of a structure, including both static and dynamic action, to the static wind-induced response.

Buildings and other adjacent structures can affect the wind forces. Wind tunnel tests may be considered as a method to improve wind force predictions or to validate design coefficients in the alternative design approach provided in Section 3.5.3.

3.3.2.2 - Design for Wind
The guideway superstructure should be designed for wind-induced horizontal, \( F_h \), and vertical, \( F_v \), drag loads acting simultaneously. The wind should be considered to act on a structure curved in plan, in a direction such that the resulting force effects are maximized. For a structure that is straight in plan, the wind direction should be taken perpendicular to the longitudinal axis of the structure.

The following uniformly distributed load intensities may be used for design:

\[ F_h = \text{the greater of 50 lb/ft}^2 (2.4 \text{ kPa}) \text{ or } 300 \text{ lb/ft} (4.4 \text{ kN/m}) \]

and

\[ F_v = 15 \text{ lb/ft}^2 (0.7 \text{ kPa}) \]

The wind loads, \( F_h \) and \( F_v \), should be applied to the exposed areas of the structure and vehicle in accordance with the provisions of sections 4.3 and 4.4.

These loads and provisions are consistent with the recommendations of the AASHTO Standard Specifications for Highway Bridges derived from wind velocities of 100 mph (160 km/h). Wind loads may be reduced or increased in the ratio of the square of the design wind velocity to the square of the base wind velocity, provided that the maximum probable wind velocity can be ascertained with reasonable accuracy, or provided that there are permanent features of the terrain that make such changes safe and are viable.

The substructure should be designed for wind-induced loads transmitted from the superstructure and wind loads acting directly on the substructure. Loads for wind directions both normal to and skewed to the longitudinal centerline of the superstructure should be considered.

3.3.2.3 - Alternative Wind Load
The alternative wind load method may be used in lieu of that given in Section 3.3.2.1. Alternative wind loads are suggested for projects involving unusual height guideways, unusual gust conditions, or guideway structures that are, in the judgment of the engineer, more streamlined than highway structures.

3.3.2.3.8 The wind load per unit exposed frontal area of the superstructure, \( WS \), and of the vehicle, \( WL \), applied horizontally, may be taken as:

\[ F_h = qC_vC_gC_d \] (3-4)

Similarly, the wind load per unit exposed plan deck or soffit area applied vertically, upwards or downwards, shall be taken as:

\[ F_v = qC_vC_gC_d \] (3-5)

Where, \( C_d = 1.0 \) and \( C_v, C_g \) and \( q \) are defined in Section 3.3.2.4. The maximum vertical wind velocity may be limited to 30 mph (50 km/h).

In the application of \( F_v \) as a uniformly distributed load over the plan area of the structure, the effects of a possible eccentricity should be considered. For this purpose, the same total load should be applied as an equivalent vertical
line load at the windward quarter point of the superstructure.

3.3.2.4 -Reference Wind Pressure

The reference wind pressures at a specific site should be based on the hourly mean wind velocity of a 75-year return period. A 10-year return period may be used for structures under construction.

The reference wind pressure, \( q \), may be derived from the following expression:

\[
q = \rho \frac{V^2}{2g} \text{ lb/ft}^2
\]

where

\( V = \text{mean hourly velocity of wind, ft/sec (m/s)} \)
\( \rho = \text{density of air at sea level at 32 F (0 C)} \)
\( = 0.0765 \text{ lb/ft}^3 (1.226 \text{ kg/m}^3) \)
\( g = 32.2 \text{ ft/sec}^2 (9.807 \text{ m/s}^2) \)

For structures that are not sensitive to wind-induced dynamics, which include elevated guideways and special structures up to a span length of 400 ft (122m), the gust effect coefficient \( C_g \) may vary between 1.25 and 1.50. For design purposes, a factor of 1.33 may be used for \( C_g \). For structures that are sensitive to wind action, \( C_g \) should be determined by an approved method of dynamic analysis or by model testing in a wind tunnel. For guideway appurtenances, such as sign posts, lighting poles, and flexible noise barriers, \( C_g \) may be taken as 1.75.

The exposure coefficient or height factor, \( C_e \), may be computed from:

\[
C_e = \frac{1}{2} \left( \frac{H}{H_0} \right)^{5/2}, \text{ for } H \text{ in ft}
= \frac{5}{8} \left( \frac{H}{H_0} \right)^{5/2}, \text{ for } H \text{ in m}
\]

\( H \) is the height from ground level to the top of the superstructure. It should be measured from the foot of cliffs, hills, or escarpments when the structure is located on uneven terrain, or from the low water level when the structure is located over bodies of water. Where excessive funneling may be caused by the topography at the site, \( C_e \) should be increased by 20 percent.

The drag coefficient or shape factor, \( C_d \), is a function of many variables, the most important of which are the skew angle (horizontal angle of wind), and aspect ratio (ratio of length to width of structure). For box or I-girder superstructures and solid-shaft piers with wind acting at zero skew and pitch angles, \( C_d \) may vary between 1.2 and 2.0. A factor of 1.50 for \( C_g \) may be used for design purposes. For unusual exposure shapes, the drag coefficient, \( C_d \), should be determined from wind-tunnel tests.

Where wind effects are considered at a skew angle of \( \theta \) degrees measured from a line perpendicular to the longitudinal axis of a structure, then \( C_d \) should be multiplied by \( 0.0078 \) for the longitudinal wind load component and by \( (1 - 0.0001813 \theta^2) \) for the transverse or perpendicular load component.

3.3.2.5- Wind Load on Slender Elements and Appurtenances

Slender elements, such as light and sign supports, should be designed for horizontal wind loads provided for in Sections 3.3.2.3 and 3.3.2.4, as well as lateral and crosswind load effects caused by vortex shedding. Both serviceability and strength considerations should be investigated. Details that may cause stress concentrations due to fatigue or resonance should be avoided.

The wind drag coefficient, \( C_d \), for sign and barrier panels with aspect ratios of up to 1.0, of 1.0 to 10.0, or more than 10.0, should be 1.1, 1.2, or 1.3, respectively. For light fixtures and sign supports with rounded surfaces, octagonal sections with sharp comers, or rectangular flat surfaces, the values of \( C_d \) should be 0.5, 1.2, or 1.4, respectively. A Value of 1.2 for \( C_d \) should be used for suspended signal units.

When ice accretion is expected on the surface of slender components, the total frontal area should include the thickness of ice.

The dynamic effects of vortex shedding should be analyzed and the stress limits for \( 2 \times 10^6 \) cycles of loading shall be applied.

3.3.3 -Loads Due to Ice Pressure, ICE

Floating ice forces on piers and exposed pier caps should be evaluated according to the local conditions at the site. Consideration should be given to the following types of ice action on piers erected in bodies of water:

- Dynamic ice pressure due to ice sheets and ice floes in motion caused by stream or current flow and enhanced by wind action,
- Static ice pressure caused by thermal action on continuous stationary ice sheets over large bodies of water,
- Static pressure resulting from ice jams at a guideway site,
- Static uplift or vertical loads due to ice sheets in water bodies of fluctuating level.

Ice loads resulting from freezing rain or con-
solidation of compact snow on the guideway superstructure and vehicle should be included, as appropriate.

### 3.3.4- Loads due to Stream Current, SF

#### 3.3.4.1 - Longitudinal Loads

The load acting on the longitudinal axis of a pier due to flowing water may be computed by the following expression.\(^3\)\(^5\)

\[
SF = \frac{1}{2} C_D A h V^2 \gamma
\]

where

- \(A\) = Exposed area of the pier perpendicular to the direction of stream flow, \(ft^2\) (\(m^2\))
- \(Y\) = Mass density of water, 62.4 \(lb/ft^3\) (1000 \(kg/m^3\))
- \(V\) = Speed of stream flow, \(ft/sec\) (\(m/set\))
- \(C_D\) = 0.7 for semicircular-nosed piers
- = 0.8 for wedge-nosed piers
- = 1.4 for squared-ended piers and against drift lodged on the pier

#### 3.3.4.2 - Transverse Loads

The lateral load on a pier shaft due to stream flow and drift should be resolved from the main direction of flow. The appropriate component should be applied as a uniformly distributed load on the exposed area of the pier, below the high water level, in the direction under consideration.

### 3.4 - Loads due to Volumetric Changes

#### 3.4.1- General

Provisions should be made for all movements and forces that can occur in the structure as a result of shrinkage, creep, and variations in temperature. Load effects that may be induced by a restraint to these movements should be included in the analysis. These restraints include those imposed during construction on a temporary basis and those imposed by the rail-fastener interaction on an on-going basis. Effects due to thermal gradients within the section should also be considered.\(^3\)\(^5\)

#### 3.4.3 - Temperature, \(T\)

##### 3.4.2.1 - Temperature Range

The minimum and maximum mean daily temperatures should be based on local meteorological data for a 50-year return period. The range of effective temperature for computing thermal movements of the concrete structure should be the difference between the warmest maximum and the coldest minimum effective temperatures, which may be considered to be 5 \(F\) (2.5 \(C\)) above or below the mean daily minimum and maximum temperatures. If local temperature data are not available, the structure may be designed for a minimum temperature rise of 30 \(F\) (17 \(C\)) and a minimum temperature drop of 40 \(F\) (23 \(C\)) from the installation temperature.

#### 3.4.2.2 - Effective Construction Temperature

If the guideway is to be designed to accommodate continuously welded rails, an effective construction temperature should be selected. This temperature, which should be based on the mean daily temperature prevalent for the locality under consideration, is used to establish the baseline rail force.

#### 3.4.2.3 - Thermal Gradient Effects

Curvature caused by a temperature gradient should be considered in the design of the structure.

The temperature differential between top and bottom surfaces varies nonlinearly according to the depth and exposure of the structural elements and their locality. A winter differential of 15 \(F\) (8 \(C\)) and a summer differential of 25 \(F\) (14 \(C\)) between the top of the deck and the soffit of the structure may be used. The temperature differential should be increased in regions with high solar radiation; NCHRP 267 document may be used as a guide in this respect.\(^3\)\(^12\)

#### 3.4.2.4 - Coefficient of Thermal Expansion

In lieu of a more precise value, the coefficient of linear thermal expansion for normal weight concrete may be taken as 6.5 \(x 10^{-6}/deg F\) (12 \(x 10^{-6}/deg C\)).

#### 3.4.3-Rail-Structure Interaction \(F_R\) and \(F_r\)

Continuously welded rail directly fastened to the guideway, induces an axial force in the structure through the fastener restraint when the structure expands or contracts due to variations in temperature. Continuously welded rail is assumed to be installed in a zero stress condition at an effective installation temperature, \(T_0\). If the CWR is installed at a temperature that is different from the effective installation temperature, then the rail is physically stressed to be compatible with the zero stress condition for which it is designed at the installation temperature.\(^3\)\(^6\)

##### 3.4.3.1 - Thermal Rail Forces

Axial rail stress \(f_r\) due to a change in the temperature after installation, is expressed by

\[
f_r = E_r \alpha (T_1 - T_0)
\]

\(E_r\) = coefficient of thermal expansion
\(T_0\) = the installation temperature (zero-stress condition)
$T_f$ = the final rail temperature

$E_r$ = modulus of elasticity of rail steel, given in Section 5.6.3

For a temperature decrease, $T_f$ may be taken as the minimum effective temperature described in Section 3.4.2.1. For a temperature rise, $T_f$ may be taken as the maximum effective temperature plus 20 F (12 C). The corresponding rail force, $F_r$, is expressed by:

$$F_r = \Sigma A_f \cdot \Sigma A_r \cdot \alpha (T_f - T_0)$$ (3-10)

where $\Sigma$ implies that the forces in all rails should be summed up. The movement of the structure through the fasteners induces either a tensile or compressive axial force on the rail, depending on whether the temperature rises or drops, respectively, from that at installation.

A vertically or horizontally curved structure experiences a radial force resulting from the thermal rail forces. This radial force per unit length of rail is expressed as

$$F_r = \frac{F_r}{R}$$ (3-11)

where $R$ is the radius of curvature. $F_r$ always occurs in combination with $F_r'$.

The preceding expressions apply where there is no motion of the rail relative to the structure. Where rail motion may occur, the relaxation of the rail must be analyzed to determine its effect on the structure. Rail motion may occur when:

- Rail expansion joints are present or,
- Radial or tangential movements of rail and guideway structure at curves occur, or
- A rail break takes place, or
- Continuous rails cross structural joints, or
- Creep and shrinkage strains in prestressed concrete elements continue to take place.

**3.4.3.2 - Broken Rail Forces**

At very low temperatures, the probability of a rail break increases. The most likely place for a rail break to take place is at an expansion joint in the structure. A rail break at this location generally creates the largest forces in the structure.

When the rail breaks, it slips through the fasteners on both sides of the break until the tensile force in the rail before the break is counteracted by the reversed fastener restraint forces. The unbalanced force from the broken rail is resisted by both the unbroken rails and the guideway support system in proportion to their relative stiffnesses. The probability that more than one rail will break at the same time is small, and is generally not considered in the design.

**3.4.3.3 - Rail Gap**

The relative stiffness of the system should be proportioned so that the magnitude of the gap between broken rail ends be equal to the maximum allowable in order to prevent vehicle derailment. Typically acceptable rail gaps are in the range of 2 in. (50 mm) for a 16-in. (0.4-m) diameter wheel and up to 4 in. (100 mm) for larger wheels. Rail gap is controlled by the spacing and stiffness of the fasteners.

**3.4.4 - Shrinkage in Concrete, SH**

Shrinkage is a function of number variables, the most significant of which are the characteristics of the aggregates, the water-cement ratio of the mix, the type and the duration of curing, surface-to-volume ratio of the member, the ambient temperature and relative humidity at the time of placing the concrete. For a major transit project, shrinkage and creep behavior of the concrete mix should be validated as part of the design process. For precast members, only the portion of shrinkage or creep remaining after the element is integrated into the structure needs to be considered.

In the absence of more accurate data or method of analysis, shrinkage strain $r$-days after casting of normal weight concrete may be computed from the following expression:

$$\epsilon_{sh} = k_r k_{sh} \epsilon_{shu}$$ (3-12)

where the ultimate shrinkage strain, $\epsilon_{shu}$, is expressed as:

$$\epsilon_{sh} = 550 \left[ 1 - \left( \frac{H}{100} \right)^2 \right] \times 10^{-6}$$ (3-13)

For $0 \leq r \leq 12$ in. (300 mm),

$$k_r = \left[ 1 - \frac{r}{12} \right]^2 + 0.5, \quad (3-14)$$

where $r_j$ is in inches,

$$k_r = \left[ 1 - \frac{r_j}{300} \right]^2 + 0.5,$$

where $r_j$ is in mm.
For \( r_v > 12 \) in. (300 mm)

\[ k_v = 0.5 \]

where \( r_v \) = volume-to-surface-area ratio, \( t \) is the time in days after the end of curing, and \( H \) is the relative ambient humidity, in percent.

\[ k_v = 1 - e^{-0.10\sqrt{t}} \quad (3-15) \]

3.4.5-Creep in Concrete, CR

Creep is a function of relative humidity, volume-surface ratio and of time \( t \) after application of load. Creep is also affected by the amount of reinforcement in the section, the magnitude of sustained prestress force, the age of the concrete when the force is applied, and the properties of the concrete mix. If the design is sensitive to volumetric change, then an experimental validation of creep behavior, based on the ingredients to be used, may be necessary.

In the absence of more accurate data and procedure, creep at \( r \)-days after application of load may be expressed in terms of the initial elastic strain, \( \varepsilon_{el} \), from:

\[ \varepsilon_{cr} = \varepsilon_{el} k_r k_t \quad (3-16) \]

where,

\[ k_r = 4.250 - 0.025 H \]

For \( 0 \leq r_v \leq 10 \) in. (250 mm)

\[ k_v = \left[ 1 - \frac{r_v}{10} \right] + 0.7, \quad (3-17) \]

where \( r_v \) is in inches,

\[ k_v = \left[ 1 - \frac{r_v}{250} \right] + 0.7, \quad (3-17) \]

where \( r_v \) is in mm

For \( r_v > 10 \) in. (250 mm)

\[ k_v = 0.7 \]

where \( t \) is the time in days after application of load or prestress, and,

\[ k = 1 - e^{-0.08\sqrt{t}} \quad (3-18) \]

3.5 - Exceptional Loads

3.5.1 - Earthquake Effects, EQ

In regions designated as earthquake zones, structures should be designed to resist seismic motions by considering the relationship of the site to active faults, the seismic response of the soils at the site, and the dynamic response characteristics of the total structure in accordance with the latest edition of AASHTO "Standard Specifications for Highway Bridges." Certain local jurisdictions have Zone 4 high seismic risk requirements for analysis and design. For structures in this zone, a dynamic analysis is recommended.

3.5.2 - Derailment Load, DR

Derailment may occur when the vehicle steering mechanism fails to respond on curves or when the wheels jump the rails at too large a pull-apart gap, which may be the result of a break in a continuously welded rail. Derailment may also be caused by intervehicle collision. For the design of the top slab and the barrier wall of the guideway, both the vertical and horizontal derailment loads may be considered to act simultaneously.

The force effects caused by a single derailed standard vehicle should be considered in the design of the guideway structure components. These effects, whether local or global, should include flexure, shear, torsion, axial tension or compression, and punching shear through the deck. The derailed vehicle should be assumed to come to rest as close to the barrier wall as physically possible to produce the largest force effect. In the design of the deck slab, a dynamic load allowance of 1.0 should be included in the wheel loads.

The magnitude and line of action of a horizontal derailment load on a barrier wall is a function of a number of variables. These include the distance of the tracks from the barrier wall, the vehicle weight and speed at derailment, the flexibility of the wall, and the frictional resistance between the vehicle and the wall. In lieu of a detailed analysis, the barrier wall should be designed to resist a lateral force equivalent to 50 percent of a standard vehicle weight distributed over a length of 15 ft (5 m) along the wall and acting at the axle height. This force is equivalent to a deceleration rate of 0.5 g.

Collision forces between vehicles result from the derailment of a vehicle and its subsequent resting position against the guideway sidewall.
This eccentric load on the guideway causes torsional effects, which should be accounted for in the design. The magnitude and eccentricity of this vertical collision load is a function of the distance of the guideway center line from the side wall, the axle width and the relative position of the center lines of the car body and the truck after the collision.

3.5.3 - Broken Rail Forces, BR
Forces on the guideway support elements due to a broken rail are discussed in Section 3.4.3, under Rail-Structure Interaction.

3.5.4 - Collision Load, CL
Piers or other guideway support elements that are situated less than 10 ft (3 m) from the edge of an adjacent street or highway should be designed to withstand a horizontal static force of 225 kips (1000 kN), unless protected by suitable barriers. The force is to be applied on the support element, or the protection barrier, at an angle of 10 deg from the direction of the road traffic and at a height of 4 ft (1.20 m) above ground level. The Collision Load need not be applied concurrently with loads other than the dead load of the structure.

The possibility of overheight vehicles colliding with the guideway beam should be considered for guideways with less than 16.5 ft (5.0 m) clearance over existing roadways.

3.6 - Construction Loads
3.6.1 - General
Loads due to construction equipment and materials that may be imposed on the guideway structure during construction should be accounted for. Additionally, transient load effects during construction due to wind, ice, stream flow and earthquakes should be considered with return periods and probabilities of single or multiple occurrences commensurate with the expected life of the temporary structure or the duration of a particular construction stage.

3.6.2 - Dead Loads
Dead loads on the structure during construction should include the weights of formwork, falsework, fixed appendages and stored materials. The dead weights of mobile equipment that may be fixed at a stationary location on the guideway for long durations shall also be considered. Such equipment includes lifting and launching devices.

3.6.3 - Live Loads
Live loads on the structure during construct-

REFERENCES


CHAPTER 4 - LOAD COMBINATIONS AND LOAD AND STRENGTH REDUCTION FACTORS

4.1 - Scope
This chapter specifies load factors, strength reduction factors, and load combinations to be used in serviceability and strength designs. Structural safety is used as the acceptance criterion. The derivation of load and strength reduction factors is based on probabilistic methods, using available statistical data and making certain basic assumptions.

4.2 - Basic Assumptions
The economic life of a transit guideway is taken as 75 years. Load and resistance models were developed accordingly.

Guideway structures should meet the requirements for both serviceability and strength design. Serviceability design criteria were derived by elastic analysis; stresses and section resistances were determined accordingly. Strength design criteria were also derived by elastic analysis. However, while stresses were determined accordingly, section resistances were determined by inelastic behavior.

The load and resistance models used in this study were based on available test data, analytical results, and engineering judgment.

Live load is defined by a fully loaded standard vehicle. The weight of vehicles should include an allowance for potential weight growth. Resistance models take into account the degree of quality control during casting. Thus, the properties of factory-produced members are considered more reliable than those of cast-in-place members. Some requirements for concrete strength control specified by AASHTO are more stringent than those specified by ACI. However, ACI specifications are generally assumed in this document.

Safety is measured in terms of the reliability index. A higher reliability index, reflects a lower probability of failure. A target reliability index of 4.0 is adopted for strength design. This implies that a transit structure would have a lower probability of failure than a highway bridge, where a reliability index of 3.5 is commonly used. The higher target value is justified by the fact that the consequences of failure of a transit guideway would be far greater than those of a highway bridge. The target reliability index adopted for serviceability design, is 2.5 for cracking and 2.0 for fatigue.

The objective in deriving reliability-based load factors is to provide a uniform safety level to load-carrying components. The uncertainties in methods of analysis, material properties and dimensional accuracies are taken into account in the derivation of strength reduction factors. Uncertainties to the magnitude of imposed loads and their mean-to-nominal ratios are accounted for in the derivation of load factors. Because of the high frequency of train passes on a guideway structure, environmental and emergency loads are combined with maximum live load. The dead load factor is set at 1.30 for both precast and cast-in-place components, consistent with the AASHTO bridge specifications and ACI 343R. The derivation of load and strength reduction factors for other load components is also based on reliability approach.

4.3 - Service Load Combinations
Four service load combinations, S1, S2, S3, and S4 are listed in Table 4.3. When warranted, more load combinations may be used on specific projects. Load and strength reduction factors are not used for serviceability design.

4.4 - Strength Load Combinations
4.4.1 - General Requirements
For strength design, the factored strength of a member should exceed the total factored load effect. The factored strength of a member or cross section is obtained by taking the nominal member strength, calculated in accordance with Chapter 6, and multiplying it by the appropriate strength reduction factor, given in Section 4.4.3. The total factored load effect should be obtained from relevant strength combination, U, incorporating the appropriate load factors given in Table 4.4.

Simultaneous occurrence of loads is modeled by using available data. For the purposes of reliability analysis, loads are divided into categories according to their duration and the probability of

Table 4.3 - Service load combinations

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Expression</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>D + L + I + PS + LF + (CF or HF or Ff)</td>
</tr>
<tr>
<td>S2</td>
<td>S1 + [0.3 (WL + WS) or ICE or SF]</td>
</tr>
<tr>
<td>S3</td>
<td>S2 + T + SH + CR</td>
</tr>
<tr>
<td>S4</td>
<td>PS + D + (WS or EQ) + T + SH + CR</td>
</tr>
</tbody>
</table>
their joint occurrence, as follows:

- Permanent loads: dead load, earth pressure, structural restraint
- Gradually varying loads: prestressing effects, creep and shrinkage, differential foundation settlement, and temperature effects
- Transitory loads: live load (static and dynamic) and wind,
- Exceptional loads: earthquake, emergency braking, broken rail, derailment, vehicle collision

It is assumed that gradually varying loads act simultaneously with permanent loads. The former are taken at their maximum or minimum level, whichever yield the worse case scenario for structural performance, for the duration considered. 

Transitory and exceptional loads are combined according to Turker's rule. This rule stipulates that the maximum total load occurs when one of the load components is at its maximum value, simultaneously with the other load components taken at their average values. All possible combinations are considered in order to determine the one which maximizes the total effect. The load factors corresponding to the time-varying load combinations reflect the reduced likelihood of simultaneous occurrence of these loads.

4.4.2 - Load Combinations and Load Factors

Load combinations, together with the corresponding factors for strength design, are listed in Table 4.4. Values of load components are specified in Chapter 3.

4.4.3 - Strength Reduction Factors, $\phi$

The capacity of a section should be reduced by a strength reduction factor, $\phi$, as follows:

- For flexure only, or flexure with axial load in precast concrete $\phi = 0.95$
- For flexure only, or flexure with axial load in cast-in-place concrete $\phi = 0.90$
- For shear and torsion $\phi = 0.75$
- For axial tension $\phi = 0.85$
- For compression in members with spiral reinforcement $\phi = 0.75$
- For compression in other members $\phi = 0.70$

For low values of axial compression, $\phi$ may be increased linearly to 0.90 or 0.95 for cast-in-place or precast concrete, respectively, as the axial load decreases from 0.10 $f'_c A_g$ to zero.

The $\phi$ factors were computed with the assumption that precast concrete guideway components, with bonded post-tensioning tendons are used.

### Table 4.4 — Design Load Combinations and Load Factors

<table>
<thead>
<tr>
<th>Load component</th>
<th>U0</th>
<th>U1</th>
<th>U2</th>
<th>U3</th>
<th>U4</th>
<th>U5</th>
<th>U6</th>
</tr>
</thead>
<tbody>
<tr>
<td>D</td>
<td>1.3*</td>
<td>1.3*</td>
<td>1.3*</td>
<td>1.3*</td>
<td>1.3*</td>
<td>1.3*</td>
<td>1.3*</td>
</tr>
<tr>
<td>L, I and either CF or HF</td>
<td>1.7</td>
<td>1.4</td>
<td>1.4</td>
<td>1.4</td>
<td>1.4</td>
<td>1.4**</td>
<td></td>
</tr>
<tr>
<td>SH and CR</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>PS</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>WL + WS</td>
<td>1.5</td>
<td></td>
<td>1.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>WS</td>
<td></td>
<td>1.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ICE, T, SF, or EQ</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1.5</td>
</tr>
<tr>
<td>LF_e</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1.4</td>
</tr>
<tr>
<td>BR (F_h, F_r)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1.2</td>
</tr>
<tr>
<td>CL</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1.3</td>
</tr>
<tr>
<td>DR</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1.4</td>
</tr>
</tbody>
</table>

1. Use 0.9 when effect is more conservative.
2. * Use the weight of an empty train only.
REFERENCES*


*For recommended references, see Chapter 8.

CHAPTER 5 - SERVICEABILITY DESIGN

5.1 - General
This chapter covers the performance of reinforced concrete guideways (both prestressed and non-prestressed) under service loadings. Serviceability requirements to be investigated include stresses, fatigue, vibration, deformation and cracking.

Fatigue is included in serviceability design since high cyclic loading influences the permissible design stresses. Load combinations for serviceability design are given in Section 4.3. Durability considerations are given in Section 2.3.6.

5.2 - Basic Assumptions
Force effects under service loads should be determined by a linear elastic analysis. For investigation of stresses at service conditions, the following assumptions are made:

a. Strains are directly proportional to distance from the neutral axis
b. At cracked sections, concrete does not resist tension
c. Stress is directly proportional to strain.

5.3 - Permissible Stresses
5.3.1 - Non-prestressed Members
Fatigue and cracking are controlled by limiting the stress levels in the concrete and the non-prestressed reinforcement. The stress limitations are discussed in Sections 5.5 and 5.8.

5.3.2 - Prestressed Members
5.3.2.1 - Concrete
Flexural stresses in prestressed concrete members should not exceed the following:

(a) At transfer:

Stresses before losses due to creep, shrinkage and relaxation and before redistribution of force effect take place, should not exceed the following:

- Compression
  1. pretensioned members: $0.60f_{ct}'$
  2. post-tensioned members: $0.55f_{ct}'$

- Tension in members without bonded non-prestressed reinforcement in the tension zone: $0.40f_{ct}'$

In the absence of more precise data, the cracking stress of concrete, $f_{ct}'$, may be taken as $7.5\sqrt{f_{ct}}$ (psi) ($0.6\sqrt{f_{ct}}$ MPa).

- Tension in members with bonded non-prestressed reinforcement in the tension zone: $1.00f_{ct}'$

Where the calculated tensile stress is between $0.40f_{ct}'$ and $1.00f_{ct}'$, reinforcement should be provided to resist the total tensile force in the concrete computed on the basis of an uncracked section. The stress in the reinforcement should not exceed $0.60f_{p}$ or 30 ksi (200 MPa), whichever is smaller.

- Tension at joints in segmental members:

  - Without bonded non-prestressed reinforcement passing through the joint in the tension zone: $0.0$
With bonded non-prestressed reinforcement passing through the joint in the tension zone:

\[ 0.40f'_{cr} \]

Where the calculated tensile stress is between zero and \( 0.40f'_{cr} \), reinforcement should be provided to resist the total tensile force in the concrete computed on the basis of an uncracked section. The stress in the reinforcement should not exceed \( 0.60f'_y \) or 30 ksi (200 MPa), whichever is smaller.

(b) Service loads:

Stresses, after allowance for all losses due to creep, shrinkage and relaxation and redistribution of force effects, should not exceed the following:

- Compression:
  - Load combination S1 or S2,
    - Precast members: \( 0.45f'_c \)
    - Cast-in-place members: \( 0.40f'_c \)
  - Load combination S3 or S4,
    - Precast members: \( 0.60f'_c \)
    - Cast-in-place members: \( 0.55f'_c \)

- Tension in precompressed tensile zones:
  - For severe exposure conditions, such as coastal areas, members in axial tension, and load combination S3 and (S4) moderate case applies: 0.0
  - For moderate exposure conditions, and for load combination S2: \( 0.40f'_c \)

In the absence of more precise data, the cracking stress of concrete, \( f'_{cr} \) may be taken as \( 7.5 \sqrt{f'_c} \) (psi) (0.6 \sqrt{f'_c} MPa).

- Other cases and extreme operating conditions at load combinations S3 and S4: \( 0.80f'_{cr} \)
  - For segmental members without bonded prestressed reinforcement passing through the joints: 0.0
  - For design against fatigue: 0.0
  - Tension in other areas should be limited by allowable stresses at transfer.

5.3.2.2 - Steel

The stress in prestressing steel should not exceed the values given in Table 5.3.

The maximum stress at jacking should in no case exceed \( 0.94f'_{pu} \), or the maximum value recommended by the manufacturer of the prestressing tendons and anchorages, while that at transfer should in no case exceed \( 0.82f'_{pu} \). The maximum stress in the post-tensioning tendons, at anchorages and couplers, immediately after tendon anchorage should not exceed \( 0.70f'_{pu} \) in accordance with ACI 318R.

5.3.3 - Partial Prestressing

The preceding tensile strength limitations may be waived if calculations, based on approved or experimentally verified rational procedures, demonstrate adequate deflection, cracking and fatigue control under specified loading combinations.

<table>
<thead>
<tr>
<th>Type of Steel</th>
<th>Prestressing Stage</th>
<th>At jacking:</th>
<th>At transfer:</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Stress-relieved strand and wire, ( f'<em>{pu} = 0.85 f'</em>{pu} )</td>
<td>Low relaxation strand and wire, ( f'<em>{pu} = 0.90 f'</em>{pu} )</td>
<td>High strength bar ( f'<em>{pu} = 0.80 f'</em>{pu} )</td>
</tr>
<tr>
<td>Prestressing Stage</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pretensioning</td>
<td>0.80 ( f'_{pu} )</td>
<td>0.80 ( f'_{pu} )</td>
<td>0.75 ( f'_{pu} )</td>
</tr>
<tr>
<td>Post-tensioning</td>
<td>0.70 ( f'_{pu} )</td>
<td>0.74 ( f'_{pu} )</td>
<td>0.66 ( f'_{pu} )</td>
</tr>
<tr>
<td>Cast-in-place members</td>
<td>0.80 ( f'_{pu} )</td>
<td>0.85 ( f'_{pu} )</td>
<td>0.75 ( f'_{pu} )</td>
</tr>
<tr>
<td></td>
<td>0.70 ( f'_{pu} )</td>
<td>0.74 ( f'_{pu} )</td>
<td>0.66 ( f'_{pu} )</td>
</tr>
</tbody>
</table>
5.4 - Loss of Prestress

In determining the effective prestress, allowance should be made for the following sources of prestress loss:

a. slip at the anchorage
b. friction losses due to intended and unintended (wobble) curvature in the tendons
c. elastic shortening of concrete
d. creep of concrete
e. shrinkage of concrete
f. relaxation of steel

The amount of prestress loss due to these causes depends on a number of factors that include, properties of the materials used in the structure, the environment, and the stress levels at various loading stages. Accurate estimates of prestress loss require recognition that the individual losses resulting from the above sources are interdependent.

The losses outlined above may be estimated using the methods outlined in the AASHTO bridge specifications, ACI 343R, or References 5.1 through 5.3.

For preliminary design of structures, using normal density concrete, the lump sum losses shown in Table 5.4 may be used. Lump sum losses do not include anchorage and friction losses in post-tensioned tendons. The losses are higher than those in the AASHTO bridge specifications due to the higher jacking stresses.

For members constructed and prestressed in multiple stages, or for segmental construction, the stress level at the commencement and termination of each stage should be considered.

5.5 - Fatigue

5.5.1 - General

A transit guideway may undergo six million or more vehicle passes at various load levels during its lifetime. This may be equivalent to three to four million cycles at maximum live load level. Such high levels of cyclic loading render guideways prone to fatigue failure.

Areas of concern are the prestressing steel and the reinforcing bars located at sections where a large number of stress cycles may occur at cracked sections.

5.5.2 - Concrete

Under service load combination S1, the flexural compressive stress in concrete, should not exceed $0.45f'_c$ at sections where stress is cyclic and no tensile stresses are allowed.

5.5.3 - Non-prestressed Reinforcement

Under service load condition S1, the stress range in straight flexural reinforcing bars $f_{fr}$ and $f_{sr}$, in accordance with AASHTO bridge specifications, should not exceed the following:

For straight bars:

$$f_{fr} = (21 - 0.33f_m + 8 \cdot r/h), \text{ ksi (5-1)}$$

$$= (145 - 0.33f_m + 55 \cdot r/h), \text{ MPa}$$

where,

$$r/h = \text{radius-to-height ratio of transverse deformations. When actual value is not known, r/h = 0.3}$$

$$f_m = \text{algebraic minimum stress, ksi (MPa)} \text{ (tension, positive; compression, negative)}$$

For bent flexural bars, stirrups and bars containing welds conforming to requirements of AWS D1.4:

$$f_{sr} = 0.50f_{fr} \text{ (5-2)}$$

Bends and welds in principal reinforcement should not be used in regions of high stress range.

For shear reinforcement, the change in stress, $f_{sv}$ may be computed as follows:

$$f_{sv} = \frac{\Delta V}{Ajd}, \text{ksi (MPa) (5-3)}$$

### Table 5.4- Lump Sum Losses for Preliminary Design^{512}

<table>
<thead>
<tr>
<th></th>
<th>PRETENSIONED</th>
<th></th>
<th>POST-TENSIONED</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Stress relieved</td>
<td>Low relaxation</td>
<td>Stress relieved</td>
<td>Low relaxation</td>
</tr>
<tr>
<td>At transfer</td>
<td>29 (200)</td>
<td>19 (130)</td>
<td>4 (30)</td>
<td>4 (30)</td>
</tr>
<tr>
<td>After transfer</td>
<td>37 (255)</td>
<td>22 (150)</td>
<td>37 (255)</td>
<td>20 (135)</td>
</tr>
<tr>
<td>Total</td>
<td>66 (455)</td>
<td>41 (280)</td>
<td>41 (285)</td>
<td>24 (165)</td>
</tr>
</tbody>
</table>

Units are ksi (MPa).
where

\[ \Delta V = \text{the range of the shear force at a section, k(N)} \]
\[ s = \text{spacing of shear reinforcement, in. (mm)} \]
\[ A_s = \text{area of shear reinforcement, in.}^2 \text{ (mm') } \]
\[ jd' = \text{distance between tensile and comprehensive forces at a section based on an elastic analysis, in. (mm).} \]

For torsion reinforcement, the change in stress, \( f_{st} \), may be computed for box sections or sections where \( a/b < 0.6 \), as follows:

\[ f_{st} = \frac{\Delta T s}{(1.7 A_{sd} A_s)} \]  

where,

\[ \Delta T = \text{the range of torsion at a section, k-in. (N-mm)} \]
\[ s = \text{spacing of torsional reinforcement, in'} \]
\[ A_t = \text{area of torsional reinforcement, in'}^2/mm^2 \]
\[ A_{oh} = \text{area enclosed by the centerline of closed transverse torsional reinforcement, in'}^2/mm^2 \]
\[ a,b = \text{the shorter and longer center-to-center dimensions of closed rectangular stirrups, respectively, in. (mm).} \]

For combined effects of shear and torsion

\[ f_{sv} + f_{st} < f_{fr} \]  

5.6 - Vibration

5.6.1 - General

Vibration of the guideway during the passage of a transit vehicle induces motion of the vehicle that result in a poor ride quality. Thus guideways must be designed to provide an acceptable level of passenger comfort. This entails consideration of the vehicle-guideway interaction.

The most significant factor affecting ride quality is the acceleration level experienced by the passenger and, as a result, comfort criteria are usually expressed in terms of acceleration limits.

Maximum dynamic effects occur when the frequency of the vehicle is close to the natural frequency of the guideway, giving rise to a quasi-resonant condition. For a guideway structure, the only natural frequency which usually needs to be considered is its lowest, or fundamental, natural flexural frequency. A quasi-resonance condition may be avoided by ensuring that the structure frequency is outside the frequency range of the vehicle, as provided by the manufacturer. Thus, natural frequencies of the guideway must be investigated in the design process.

5.6.2 - Natural Frequency

The expression for the fundamental flexural frequency of a simply supported beam is given in Section 3.3.2.

The fundamental frequency of a continuous beam, having a series of equal spans, is the same as that of a simply supported beam of the same span length. For a continuous beam, in which the spans are unequal, a reasonable estimate of the fundamental frequency may be obtained by assuming the longest span to be simply-supported. A more accurate value of the fundamental frequency may be obtained using the approaches in References 5.5 and 5.6. Effects of the horizontal curvature can be accounted for as shown in Reference 5.7.

Continuous beams have frequencies of higher flexural modes which are closer to the fundamental frequency than is the case for simply supported beams. Consequently, care should be taken to ensure that one of these higher frequencies for a continuous beam does not coincide with frequency of the vehicle.

Attention should be given to torsional frequencies of the guideway and the vehicle in guideway where not all supports can resist torsional effects. Methods for the computation of torsional frequencies can be found in standard textbooks on vibrations of structures.

5.6.3 - Modulus of Elasticity

The modulus of elasticity, \( E_c \), for concrete may be taken as \( 33 \text{ ksi} \) in psi (\( 235 \text{ MPa} \) in MPa) for values of \( w_c \) between 90 and 155 lb/ft\(^3\) (1500 and 2500 kg/m\(^3\)). For normal weight concrete, \( E_c \) may be taken as \( 57,000 \text{ psi} \) (\( 400 \text{ MPa} \)).

The modulus of elasticity, \( E_p \), for non-prestressed reinforcement may be taken as \( 29,000 \text{ psi} \) (\( 200 \text{ MPa} \)).

The modulus of elasticity, \( E_p \), for prestressing tendons shall be determined by tests or supplied by the manufacturer.
5.7 -Deformation

5.7.1 - General

Deflections and rotations due to external loading, prestress, and volume changes due to temperature, creep, and shrinkage, should be considered in the design; excessive deformations can affect the structure and the ride quality directly. Of particular importance is the angular discontinuity at the guideway surface at the ends of beams at expansion joint.

Deformation in members under sustained loading should be calculated as the sum of both the immediate and the long-term deformations. Deflections, which occur immediately upon application of load, should be computed by the usual methods for elastic deflections.

5.7.2 - Non-prestressed Members

5.7.2.1 - Immediate Deflection

For simple spans the effective moment of inertia, \( I_e \), should be taken as

\[
I_e = \left( \frac{M_{cr}}{M_a} \right)^3 I_t + \left[ 1 - \left( \frac{M_{cr}}{M_a} \right)^3 \right] I_c 
\]

(5-6)

where,

\( I_t \) = moment of inertia of cracked section transformed to concrete, in\(^4\) (m\(^4\))

\( I_g \) = moment of inertia of gross concrete section about the centroidal axis, neglecting the reinforcement, in\(^4\) (m\(^4\))

\( M_a \) = maximum moment in member at stage for which deflection is being computed, lb - in. (N - mm)

\( M_{cr} \) = cracking moment = \( f_{cr} I_c / y_t \)

\( f_{cr} \) = cracking stress in concrete, psi (MPa)

\( y_t \) = distance from the centroidal axis of a cross-section (neglecting the reinforcement) to the extreme fiber in tension, in (mm).

For continuous spans, the effective moment of inertia may be taken as the average of the values obtained using the preceding equation for the critical positive and negative moment sections.

5.7.2.2 - Long-Term Deflection

In lieu of a detailed analysis, the additional long-term deflection resulting from creep and shrinkage for both normal weight and light-weight concrete flexural members may be estimated by multiplying the immediate deflection, caused by the sustained load being considered, by the factor

\[ \lambda = \frac{T}{1 + 50 \rho'} \]  

(5-7)

where,

\( \rho' \) = reinforcement ratio for non-prestressed compressive reinforcement

\( T \) = time-dependent factor for sustained load, and may be taken as:

5 years or more, \( T = 2.0 \)
12 months, \( T = 1.4 \)
6 months, \( T = 1.4 \)
3 months, \( T = 1.0 \)

5.7.3 - Prestressed Members

The effects induced by prestress should be included in the computation of deformation.

5.7.3.1 - Immediate Camber/Deflection

The moment of inertia should be taken as that of the gross concrete section.

5.7.3.2 - Long-Term Camber/Deflection

In lieu of a detailed analysis, long-term camber and deflection, as a function of instantaneous camber and deflection for members constructed and prestressed in a single stage, may be estimated by multiplying the initial camber or deflection by the factors shown in Table 5.7.

It should be noted that these factors apply to simple spans. For continuous spans, in the absence of a detailed analysis, long-term deflections may be estimated by applying two thirds of the factors given in the table.

5.8 - Crack Control

Cracking should be controlled in non-prestressed reinforced members by suitable detailing and sizing of the reinforcement. Prestressed concrete members should contain non-prestressed reinforcement at the precompressed tensile zone.

Provisions should be made in design for positive moments that may develop in the negative moment regions of precast prestressed units erected as simple span and made continuous for live loads. The effects of loading in remote spans, as well as shrinkage, creep, and elastic shortening of the piers should also be considered in the design.
Table 5.7—Suggested Multipliers to be Used as a Guide in Estimating Long-time Cambers and Deflections for Typical Members

<table>
<thead>
<tr>
<th></th>
<th>Without composite topping</th>
<th>With composite topping</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>At erection:</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Deflection (downward) component - apply to the elastic deflection due to the member weight at release of prestress</td>
<td>1.85</td>
<td>1.85</td>
</tr>
<tr>
<td>Camber (upward) component - apply to the elastic camber due to prestress at the time of release of prestress</td>
<td>1.80</td>
<td>1.80</td>
</tr>
<tr>
<td><strong>Final:</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Deflection (downward) component - apply to the elastic deflection due to the member weight at release of prestress</td>
<td>2.70</td>
<td>2.40</td>
</tr>
<tr>
<td>Camber (upward) component - apply to the elastic camber due to prestress at the time of release of prestress</td>
<td>2.45</td>
<td>2.20</td>
</tr>
<tr>
<td>Deflection (downward) - apply to elastic deflection caused by the composite topping</td>
<td>3.00</td>
<td>3.00</td>
</tr>
<tr>
<td>Deflection (downward) - apply to elastic deflection due to superimposed dead load only</td>
<td>2.30</td>
<td></td>
</tr>
</tbody>
</table>

5.8.1 - Non-prestressed Members

Tensile reinforcement should be distributed in the tension zones so that the calculated stress in the reinforcement would not exceed the following:

\[
f_s = \frac{z}{\sqrt{d_c A}} \leq 0.60f_y \tag{5-8}\]

The quantity \(z\) should not exceed 130 kips/in. (23 kN/mm) for severe exposure and 170 kips/in (30kN/mm) for other conditions; where

\[d_c = \text{thickness of the concrete cover measured from the extreme tensile fiber to the center of the bar located closest thereto.}\]

\[A = \text{effective tension area of concrete surrounding the main tension reinforcing bars and having the same centroid as that reinforcement, divided by the number of bars. When the main reinforcement consists of several bar sizes, the number of bars should be computed as the total steel area divided by the area of the largest bar used.}\]

5.8.2 - Prestressed Members

Reinforcement must be provided to control two types of cracking, namely, bursting and spalling at the anchorage zones of post-tensioned members. Several methods of proportioning the reinforcement are available. The following approach may be applied to the bursting component of cracking; it is derived from expressions in Reference 5.10, and presented in Reference 5.12 as a representative approach.

5.8.2.1 - Post-Tensioned Members

The maximum stress, \(f_{bs}\), causing bursting may be computed from

\[
f_{bs} = \Psi \frac{F_s}{A^2}, \text{ psi} \tag{5-9}\]

\[
\Psi = e^{-\frac{b_s}{z}} \tag{5-10}\]

and should not exceed \(0.80f_y + 20p_{bs}\)

where

\[\Psi\] represents the stress due to the bursting component of cracking.
\[ \rho_{bs} = \frac{A_{bs}}{b_b S} \]  

(5-11)

\[ A_{bs} = \text{area of non-prestressed reinforcement located perpendicular to a potential bursting crack, in.}^2 (\text{mm}^2) \]

\[ b_b = \text{width of concrete in the plane of a potential bursting crack, in. (mm)} \]

\[ S = \text{spacing of reinforcement to resist bursting or pitch of spiral reinforcement, in. (mm)} \]

\[ f_{cri} = \text{cracking stress of concrete at time of initial prestress, psi (MPa)} \]

For calculating \( f_{bs} \), a symmetrically placed square anchor of side \( a_1 \) acting on a square prism of side and depth \( a_2 \) may be assumed. The dimension \( a_2 \) should be the minimum distance between the centerline of anchors or two times the distance from the centerline of the anchor to the nearest edge of concrete, whichever is lesser [Fig. 5.8.2(a)]. For circular anchors, \( a_1 \) should be taken as the side of a square with an area equal to the area of the circular anchor.

The total force, \( F_{bs} \), causing bursting in a plane perpendicular to the longitudinal axis of the tendon, may be computed from

\[ F_{bs} = 0.70 F_{sji} \psi \]  

(5-12)

Reinforcement to resist the bursting force should be uniformly distributed from 0.52\( X_m \) to a distance equal to \( a_2 \), measured from the loaded face of the end block [Fig. 5.8.2(b)], where:

\[ X_m = 0.54 (1 - \psi) a_2 \]  

(5-13)

The stress in the reinforcement should not exceed 30 ksi (200 MPa) nor 0.85\( f_p \).

Reinforcement to control spalling cracks in both the horizontal and vertical planes at the anchorage zones should be provided within 0.2\( h \) of the end of the member. The spalling force may be determined by the method described in Reference 5.11. The end stirrup should be placed as closely to the end of the member as practicable with adequate cover. The reinforcement should extend over the full depth and width of the member. The stress in the reinforcement should not exceed 20 ksi (140 MPa).

5.8.2.2 - Pretensioned Members

End blocks are not required where all tendons are pretensioned strand.

Vertical stirrups to resist a tension equal to at least four percent of the prestressing force at transfer should be distributed uniformly over a length equal to 0.2\( h \) from the end of the girder. The end stirrup should be placed as closely to the end of the member as practicable. The stress in the reinforcement should not exceed 20 ksi (140 MPa).

The ends of members with flanges should be reinforced to enclose the prestressing steel in the flanges.

Transverse reinforcement should be provided in the flanges of box girders and should be anchored into the webs of the girder.

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Fig. 5.8.2(a) Symmetrical Prism Concept
Fig. 5.8.2(b) Distribution of Stress Causing Bursting

REFERENCES*


* For recommended references, see Chapter 8.

CHAPTER 6 - STRENGTH DESIGN

6.1 - General Design and Analysis Considerations

The recommendations in this chapter are intended for reinforced concrete guideways proportioned for adequate strength using load combinations, load factors, and strength reduction factors as specified in Chapter 4. The recommendations are based principally on ACI 318, “Building Code Requirements for Reinforced Concrete,” hence, may also be applied to non-prestressed components of a guideway structure, where applicable.

All members of statically indeterminate structures should be designed for the maximum effects of the specified loads as determined by 1) elastic analysis, or 2) any acceptable method that takes into account the nonlinear behavior of reinforced concrete members when subjected to bending moments approaching the strength of the member. Analysis should satisfy the conditions of equilibrium, compatibility and stability at all points in the structure and at all magni-
tudes of loading up to ultimate.

Negative moments calculated by elastic analysis at the supports of continuous pre-stressed and non-prestressed flexural members, for any assumed loading arrangement, may be increased or decreased in accordance with the provisions of ACI 318.

For guideways made continuous by post-tensioning over two or more spans, the effects of secondary moments due to the reactions induced by prestressing should be included.

Any reasonable assumption may be adopted for computing the relative flexural and torsional stiffness of members in a statically indeterminate system. The moments of inertia used to obtain the relative stiffnesses of the various members may be determined from either the uncracked concrete cross section, neglecting the reinforcement, or from the transformed cracked section, provided the same method is used throughout the analysis. The effect of variable cross sections should be considered in analysis and design.

The span length of members that are not built integrally with their supports should be the clear span plus the depth of the member. It need not exceed the distance between centers of supports. In analysis of statically indeterminate members, center-to-center distances should be used to determine moments. Moments at faces of supports may be used for design of members.

The possible buckling of a slender member or flange subject to compressive loading should be considered.

6.2 -Design for Flexure and Axial Loads
Guideways should be designed to have design strengths at all sections at least equal to the required strengths calculated for the factored loads and forces in such combination as stipulated in Chapter 4. Design strength of a member or cross section should be taken as the nominal strength calculated in accordance with requirements and assumptions of this chapter, multiplied by a strength reduction factor, $\phi$, as defined in Chapter 4. The strength design of members for flexure and axial loads should be based on the provisions of ACI 318.

6.3 -Shear and Torsion
6.3.1 -Introduction
In transit guideways, torsional moments are produced by wind load on the vehicles and on the structures, by the horizontal hunting action of the vehicles, by the centrifugal forces of the vehicles on curved tracks, and by vertical loads on curved members. These torsional effects must be combined with the shear effects in the design of reinforcement. Large shear and torsion effects may also be caused by derailment of vehicles.

Guideway structures are often made continuous to better resist the torsional effects as well as to allow more slender structures. The use of continuity, particularly with horizontal curvature, can create a shear and torsion condition that is quite complex.

6.3.2 -Conventional Design Methods
The conventional design method for shear and torsion in the United States is covered in Chapter 11 of ACI 318. This method was later adopted in the AASHTO bridge specification except that the criteria are augmented by requirements for fatigue design.

Chapter 11 of ACI 318 includes shear provisions for prestressed concrete as well as non-prestressed concrete. However, the torsion design provisions in this code are applicable only to non-prestressed concrete and not to prestressed concrete. This represents a severe limitation, because transit girders are normally prestressed. Generalized design methods, based on ACI criteria, have been proposed for prestressed concrete.

The conventional ACI method was originally formulated for building structures, in which the elements are relatively small and the cross sections are made up of rectangular components. Careful consideration must be given when this method is applied to transit guideways which are relatively large and frequently consist of thin-wall box sections or double-tee sections.

When applied to transit guideways, the conventional ACI method has the following limitation. First, this method is applicable to beams that are made up of rectangular components. It must be generalized when applied to arbitrary cross sections, such as a box girder with a trapezoidal section.

Second, in this method, the shear web reinforcement and torsion web reinforcement are simply added, resulting in a conservative design. In a large box girder, it should be possible to design for less web reinforcement for the wall where shear and torsion are additive.

Third, in the ACI method, the flexural steel and the torsional longitudinal steel are added. This simple addition of the flexural compression steel to the torsional longitudinal steel in the flexural compression zone is quite conservative. In a large transit guideway, considerable economy can be obtained when a more rigorous treatment is made.

Fourth, although the generalized ACI method is able to unify the design of prestressed and
non-prestressed concrete, the method becomes very tedious because of its empirical nature.

6.3.3 - Truss Model Approach

Design methods based on the truss model or the Compression Field Theory, provide a clear concept of how reinforced concrete elements resist shear and torsion after cracking. It allows a logical unification of shear and torsion, and is applicable to prestressed and non-prestressed concrete. The interaction of shear and torsion with bending and axial load also becomes consistent and comprehensible.

The truss model approach was first adopted by the CEB - FIB Model Code. This code has been successfully used for the design of curved box girders. It has recently gained acceptance in North American Codes.

First, the arbitrary definitions of the center line of shear flow and the wall thickness in torsion may be unconservative for relatively small elements. Second, the provisions to prevent the compression failure of the concrete diagonal struts may become unreasonable in some cases. Third, omission of torsional moment in the so-called compatibility torsion condition could cause excessive cracking.

A truss model approach was developed by Collins and Mitchell for shear and torsion design. The method uses a compression field theory and allows for the introduction of prestress forces. With some modifications, it was incorporated into CAN3-A23-3, and the method has been used in the United States and Canada.

The method contains several features. First, the omission of concrete cover is a departure from the American design practice. Second, the equation for calculating the wall thickness in torsion when relatively large percentages of web reinforcement are present may result in conservative wall thicknesses.

6.3.4 - Warping Torsion

All the torsion design provisions currently available deal with members of bulky cross sections. For such members, St. Venant torsion predominates and the warping torsional resistance can be ignored without appreciable error. However, thin-wall open sections, such as double-tees, are used in transit systems. For such structures, the working torsional resistance should be considered. The CEB Code allows for the design of warping effects to be accomplished by ensuring that equilibrium exists between each thin-wall element of the open section. Alternatively, a conservative design can be obtained by conducting an elastic analysis of the warping torsion and adding the warping stresses to the other shear and longitudinal stresses in the section.

REFERENCES*


*For recommended references, see Chapter 8.

CHAPTER 7 - REINFORCEMENT DETAILS

For nonseismic and nonfatigue design the reinforcement details should be in accordance with ACI 315 and ACI 318. For seismic design or when fatigue conditions exist, the reinforcement details given in the AASHTO bridge specifications should be used.

CHAPTER 8 - REFERENCES

8.1 - Recommended References

The documents of the various standards-producing organizations referred to in this document are listed below with their serial designation.
Specifications for Highway Bridges.

American Concrete Institute

116R Cement and Concrete Terminology

117 Standard Specifications for Tolerances for Concrete Construction and Materials

215R Considerations for Design of Concrete Structures Subjected to Fatigue Loading

315 Details and Detailing of Concrete Reinforcement

318 Building Code Requirements for Reinforced Concrete

318R Commentary on Building Code Requirements for Reinforced Concrete

318M Building Code Requirements for Reinforced Concrete

343R Analysis and Design of Reinforced Concrete Bridge Structures

358R State-of-the-Art Report on Concrete Guideways

American Railway Engineering Association
Manual of Standard Practice (AREA)

American Welding Society

D1.4 Structural Welding Code-Reinforcing Steel

Canadian Standards Association

CAN3-A23.3 Design of Concrete Structures for Buildings

CAN3-S6-M88 Design of Highway Bridges

These publications may be obtained from the following organizations:

American Association of State Highway and Transportation Officials
444 N. Capitol St., N.W., Suite 225
Washington, D.C. 20001

American Concrete Institute
P.O. Box 9094
Farmington Hills, MI 48333-9094

American Railway Engineering Association
50 F Street, N.W., Suite 7702
Washington, D.C. 20001-2183

American Welding Society
550 N.W. 42nd Avenue
Miami, FL 33126

Canadian Standards Association
178 Rexdale Blvd.
Rexdale (Toronto), Ontario
Canada M9W 1R3

This report was submitted to letter ballot of the committee and was approved in accordance with ACI balloting procedures.