VOLUME III – AERIAL GUIDEWAY

CHAPTER 3 – STRUCTURAL DESIGN CRITERIA

REVISION 1

Program Management Consultant

Submitted ___________________________ Date 4/16/2009

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3.01 GENERAL

3.01.1 PURPOSE OF THE STRUCTURAL DESIGN CRITERIA

Aerial guideway structures and their components shall be designed to meet the geometric, functional, aesthetic and structural requirements of these criteria.

3.01.2 SCOPE OF THE STRUCTURAL DESIGN CRITERIA

These criteria are intended for the design of aerial guideway and station structural elements subjected to transit vehicular loading. These criteria apply to all new construction occurring in 2005 and beyond.

The design provisions of these criteria employ the Load and Resistance Factor Design (LRFD) methodology. These criteria are minimum requirements not intended to supplant the exercise of engineering judgment by the Engineer of Record.

Reference to other Volumes and Chapters of the Compendium of Design Criteria are included herein. Where the reference calls out a Section reference only, the Section belongs to Volume III, Guideway Design Criteria. Where the reference calls out of a Volume and Section, the Section belongs to the referenced Volume.

Commentary is provided within the Structural Design Criteria to provide information to the Engineer of Record regarding the formulation of the criteria provisions. The commentary is not intended to provide a complete historical background for the criteria, nor a detailed summary of the studies and research data reviewed in establishing the criteria. However, reference to some of the research data is provided for those who wish to study the
background material in greater depth. These references and the commentary are not intended to be a part of these criteria.

**Commentary:** The Phase 1 System and the Palmetto Extension were designed using the Compendium of Design Criteria developed for Phase 1. Although the LRFD methodology has been calibrated with the inclusion of the Phase 1 system, the design criteria included herein cannot be utilized as the basis for design of these existing elements.

### 3.01.3 REFERENCE CODES

The following are reference codes used in these criteria. The current version of codes, standards and regulations shall apply, and unless otherwise directed, all addenda, interim supplements and revisions and ordinances by the respective code body shall also apply. Where conflicts exist between these requirements, and unless otherwise directed by MDT, the more stringent requirement shall take precedence.

A. AASHTO (American Association of State Highway and Transportation Officials) LRFD Bridge Design Specifications for Customary U.S. Units

B. Florida Building Code

C. ASCE (American Society of Civil Engineers) Standard 7, Minimum Design Loads for Buildings and Other Structures

D. FDOT (Florida Department of Transportation) Structures Design Guidelines
3.02 GEOMETRIC REQUIREMENTS

3.02.1 HORIZONTAL ALIGNMENT

The alignment of the guideway shall be developed to provide adequate horizontal clearances between components of the aerial guideway structure and privately or publicly owned streets, highways, railways, utility lines, and other structures or properties as specified in Section 2.06.

The coping lines of the guideway deck slab shall follow the alignment of the track, providing adequate clearances between components on the aerial guideway structure and the transit vehicle to accommodate the transit vehicle dynamic clearance envelope as specified in the MDT Directive Drawings. Where clearances require adjustment due to curvature, the guideway deck slab coping lines shall be smoothly transitioned within the transition spiral.

The webs of the guideway superstructure may be chorded and do not have to follow the alignment of the track. Horizontal offsets of the exterior webs of the guideway superstructure are highly discouraged. Where documentation is provided demonstrating that flush exterior webs cannot be provided in a cost-effective manner, special transition sections (structural or non-structural) designed to enhance appearance will be considered on a case-by-case basis, and requires prior approval from MDT before incorporating into the design.

3.02.2 VERTICAL ALIGNMENT

The alignment of the guideway shall be developed to provide adequate vertical clearances between components of the aerial guideway structure and privately or publicly owned streets, highways and railways as specified in Section 2.06.
3.02.3 TOLERANCES

The geometry of the aerial guideway structure and its components shall consider the effects of construction tolerances. Project Specifications shall clearly identify the applicable tolerances assumed in the design, which shall be utilized in the field to verify adequate as-built clearances.
3.03 SYSTEMS REQUIREMENTS

3.03.1 ELECTRIFICATION AND TRAIN CONTROL

Space shall be provided for the installation of the contact rail and coverboard system, train control and communications systems, emergency trip stations, and other electrical, mechanical, and electronic appurtenances.

3.03.2 TRACKWORK

The rails are to be attached to the guideway structure using direct-fixation rail fasteners. Adequate provision shall be made in the design of the guideway structure for the mounting and alignment of the rails. Reinforcing steel shall be designed and detailed to provide adequate clearance to the fastener anchorages at the maximum spacing specified in Section 2.05.8.

3.03.3 WALKWAY AREAS

A continuous walkway shall be provided along the aerial guideway structure. The walkway shall be accessible to passengers during an emergency evacuation and shall meet the requirements of Volume I, Section 9.03.2 (Fire/Life Safety Criteria).

3.03.4 ACOUSTIC CONTROL

The design of the aerial guideway structure shall follow the recommendations of the noise studies performed on each corridor, with regard to the implementation of measures to reduce transit noise.

Components attached to the guideway structure shall be securely tied down and shall be rattle-proof. Where necessary for noise abatement, dampening devices shall be used at supports of components subject to vibration.
3.03.5 GROUNDING AND LIGHTNING PROTECTION

The design of the aerial guideway structures shall provide for grounding and lightning protection as specified in Section 4.04.

3.03.6 FIRE PROTECTION

Design and details of aerial guideway structures shall comply with the relevant provisions of Volume I, Section 9.03.2 (Fire/Life Safety Criteria).

Fire ratings, prescribed in Volume I, Section 9.03.2 (Fire/Life Safety Criteria), are considered applicable to guideway structures. Unprotected steel girders may be used provided that the necessary approval is obtained from the Fire/Life Safety Technical Committee.
3.04 DRAINAGE REQUIREMENTS

3.04.1 GENERAL
Means for storm drainage shall be provided for all aerial guideway structures and their components in accordance with Section 2.07.

3.04.2 DRAIN DETAILS
Deck cross slopes, curbs, drainage openings in plinth pads, expansion joints, inlets, downspouts and other drainage appurtenances shall be adequately sized and detailed to drain the runoff from a 50-year design storm without exceeding a freeboard of one inch. Relief spouts shall be provided and sized to drain guideway surface without overflowing guideway edge and expansion joints when downspouts are clogged.

The superstructure shall be cambered to compensate for all long-term dead load deflections. The setting of profile grades and the cambering of bridge decks shall be designed in such a manner as to require drainage at pier locations only. Installation of a longitudinal piping system to bring storm water to the pier downspout is highly discouraged. Where documentation is provided demonstrating that guideway drainage without a longitudinal piping system cannot be provided in a cost-effective manner, longitudinal piping will be considered on a case-by-case basis, and requires prior approval from MDT before incorporating into the design.

Downspouts shall be concealed within the substructure. Inlets and downspouts shall be rattle-proof and non-corrosive. Downspouts shall be rigid, shall have a minimum inside dimension of six inches, and shall be provided with splash blocks where an underground connection to pipes or drainage structures from downspouts is not provided. Overhanging portions of concrete decks shall be provided with drip grooves.
3.05 STRUCTURAL DESIGN PHILOSOPHY AND LOAD REQUIREMENTS

3.05.1 DESIGN LIFE
All aerial guideway components and connections shall be designed for a design life of 100 years.

3.05.2 BASIS OF DESIGN
The AASHTO LRFD Bridge Design Specification shall be the basis for the structural design of all guideway superstructure and substructure components, as modified herein. The design shall meet the limit state requirements specified herein to achieve the objectives of strength, stiffness, stability, constructibility and serviceability.

Commentary: The AASHTO LRFD Bridge Design Specifications were calibrated for highway structures utilizing a statistical approach to provide an acceptable level of safety consistent with the performance of bridges subjected to automobile and truck traffic. The level of safety, measured in terms of a reliability index (β), is based on establishing an acceptable probability that the design criteria will not be exceeded. The reliability index calibrated for typical highway bridges is 3.5. This reliability index was utilized to establish the load and resistance factors used today in the design of bridges. Refer to the AASHTO LRFD Bridge Design Specifications and reference documents cited therein for more information on this calibration process.

A higher level of safety for guideway structures is justified based on the consequences of a structural failure in a public transportation system, and the lack of detour potential while re-establishing service. A reliability index of 4.0 has been established as an acceptable level of safety for transit
guideways. This reliability index results in a probability of exceeding the design criteria of less than 1/7 that of a typical highway bridge. A calibration process to establish the load and resistance factors utilized in these criteria has been developed based on statistical knowledge of loads and structural performance of the existing Phase I guideway and other similar systems. The choice of a reliability index and the calibration of the load and resistance factors for the design of transit guideways have been documented in the following references:


3.05.3 LIMIT STATE DESIGN REQUIREMENTS

All aerial guideway components and connections shall be designed in accordance with Equation 3.05.3-1 for all service limit states, fatigue and fracture limit states, strength limit states and extreme event limit states specified herein.

\[
\sum \eta_i \gamma_i Q_i \leq \phi R_n \text{ kips} \quad (\text{Eq. 3.05.3-1})
\]

in which:

\( \eta_i \) = load modifier specified in Section 3.05.5: a factor relating to ductility, redundancy and operational importance

\( \gamma_i \) = load factor specified in Section 3.05.7: multiplier applied to force effects

\( Q_i \) = force effects from the loads specified in Section 3.05.6 (kips)
\( \phi = \) resistance factor: multiplier applied to nominal resistance

\( R_n = \) nominal resistance (kips)

Force effects and displacements due to the specified design loadings shall be obtained by recognized procedures of analysis.

3.05.4 LIMIT STATE DEFINITIONS

Limit states are defined herein to establish a set of performance criteria that shall be met for given loading conditions. These loading conditions combine various loads which can occur simultaneously during operational and non-operational service. All aerial guideway components and connections shall be designed for the strength, extreme event, service and fatigue/fracture limit states as defined herein.

3.05.4.1 Strength Limit States

STRENGTH 1: Load combination relating to operational use of the guideway without wind.

STRENGTH 2: Load Combination relating to use of Owner-specified permit vehicles without wind.

**Commentary:** This limit state is provided to mirror Strength Limit State II of the AASHTO LRFD Bridge Design Specifications. Phase I criteria does not allow permit vehicles to cause force effects greater than the vehicle live load. If permit vehicles are allowed, the loads shall be applied simultaneously with other operational traffic, if applicable.
STRENGTH 3: Load Combination relating to non-operational use of the guideway with high velocity wind.

STRENGTH 4: Load Combination relating to very high dead load to live load force effect ratios.

STRENGTH 5: Load Combination relating to operational use of the guideway with operational wind.

STRENGTH 6: Load Combination relating to operational use of the guideway with emergency braking.

3.05.4.2 Extreme Event Limit States

EXTREME EVENT 1: Load Combination relating to operational use of the guideway during a seismic event for connection of superstructure to substructure only.

EXTREME EVENT 2: Load Combination relating to operational use of the guideway during a vessel or truck collision.

Commentary: Vessel collision and truck collision are considered separate events and should not be applied simultaneously.

EXTREME EVENT 3: Load Combination relating to operational use of the guideway during a derailment.

EXTREME EVENT 4: Load Combination relating to a rail fracture.
EXTREME EVENT 5: Load Combination relating to superflood (500-year) scour event.

3.05.4.3 Service Limit States

SERVICE 1: Load Combination relating to operational use of the guideway with operational wind.

SERVICE 2: Load Combination intended to control yielding of steel structures and slip of slip-critical connections due to live load.

SERVICE 3: Load Combination for longitudinal analysis relating to tension in prestressed concrete structures with the objective of crack control and to principal tension in the webs of segmental concrete girders.

SERVICE 4: Load Combination relating only to tension in prestressed concrete substructures with the objective of crack control.

SERVICE 5: Load Combination relating to non-operational use of the guideway with high velocity wind.

SERVICE 6: Load Combination relating only to control uplift and concrete tension during a derailment.

3.05.4.4 Fatigue/Fracture Limit States

FATIGUE/FRACTURE 1: Fatigue and fracture load combination relating to repetitive live load and dynamic response.
3.05.5 LOAD MODIFIERS

Load modifiers shall be in accordance with the *AASHTO LRFD Bridge Design Specifications*, except as modified below:

1. For strength limit states, the ductility load modifier, $\eta_D$, shall not be taken less than 1.00. Use of non-ductile components and connections shall require prior approval from MDT before incorporating into the design.

2. For strength limit states, the redundancy load modifier, $\eta_R$, shall not be taken less than 1.00.

3. For strength limit states, the operational importance load modifier, $\eta_I$, shall be equal to 1.00.

*Commentary:* Use of a reliability index of 4.0 in the calibration process provides for the operational importance.
3.05.6 LOADS AND LOAD DEFINITIONS

The following sustained, transient and construction loads shall be considered in the design of the aerial guideway components and connections.

Sustained Loads

- **DC** = dead load of structural components and non-structural attachments
- **DD** = downdrag force
- **DW** = dead load of cables and other utilities
- **EH** = horizontal earth pressure force
- **EL** = accumulated locked-in force effects resulting from the construction process, including the secondary forces from post-tensioning
- **ES** = earth surcharge force
- **EV** = vertical force from dead load of earth fill
- **SE** = settlement

Transient Loads

- **BU** = buoyancy
- **CF** = centrifugal force
- **CR** = creep force
- **CT** = automobile vehicular collision force
- **CV** = vessel collision force
- **DI** = derailed vehicle dynamic load
- **DR** = derailment load
- **EQ** = seismic force
- **FR** = friction force
- **HF** = horizontal dynamic load (hunting force)
- **IM** = vertical dynamic load
- **LL** = vertical standard transit vehicle load
- **LF** = longitudinal force
- **LS** = live load surcharge
- **PL** = pedestrian live load
- **RF** = rail fracture force
- **SH** = shrinkage
- **TG** = temperature gradient
- **TLR** = longitudinal thermal rail force
- **TTR** = transverse thermal rail force
- **TU** = uniform temperature
- **WA** = water load and stream pressure
- **WL** = wind on live load
- **WS** = wind on structure
Construction Loads (as applicable to concrete and steel structures)

- CLE = longitudinal construction equipment load
- CLL = distributed construction live load
- DIFF = differential load
- IE = dynamic load from equipment
- U = segment unbalance load
- WE = wind on live load
- WUP = wind uplift

The loads to be utilized for the design of the aerial guideway shall be as defined in the AASHTO LRFD Bridge Design Specification, except as modified herein.

3.05.6.1 Dead Loads: DC and DW

Dead load shall consist of the weight of all components of the structure and the superimposed dead loads it supports. Component dead load (DC) shall consist of the weight of all components of the structure. Superimposed dead load (DW) shall include the weights of all appurtenances and utilities attached to the structure, including, but not necessarily limited to, the weights of the running rails, rail fasteners, concrete rail support (plinth) pads, emergency guardrails (if provided), contact rail and coverboard with mountings and support pads, walkways, wireways, cabletrays, cables, railings and acoustical barriers. Dead loads for all elements shall account for deck camber, curvature and superelevation.

**Commentary:** The Phase I Design Criteria included estimates of the weights of all of these appurtenances. These weights were provided as a means for the Engineers of Record of system components to understand the requirements of the standard procurement contracts that were not under their control. The current methodology for splitting the construction contracts places control of all of these elements under the same Engineer
of Record; therefore, it is the Engineer of Record’s responsibility to establish these dead weights.

In the absence of more precise information, the unit weights specified in AASHTO LRFD Bridge Design Specifications Table 3.5.1-1 may be used to establish dead loads, with the exception of the following:

<table>
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<th>Material</th>
<th>Unit Weight (kcf)</th>
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<tr>
<td>South Florida Oolitic Concrete, plain</td>
<td>0.138</td>
</tr>
<tr>
<td>Ballast, including ties</td>
<td>0.150</td>
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Table 3.05.6.1-1: Unit Weights

3.05.6.2 Live Load: LL

Live load shall consist of the weight of the transit vehicle and the passengers. The vehicle length and axle spacing shall be as shown in Figure 3.05.6.2-1. Any train may consist of two, four, six or eight vehicles.

Figure 3.05.6.2-1: Transit Vehicle Configuration
The live load utilized in the load combinations specified in Section 3.05.7 shall be 120 kips distributed evenly between all axles and wheels. Reductions in live load for the various limit states, e.g., Fatigue/Fracture 1 Limit State, have been incorporated into the specified load factors.

**Commentary:** The limitation for the maximum load carried by the vehicle is based on the Maximum Guideway Design Load (MGDL) of 115.5 kips based on the existing Phase I Design Criteria. Load Rating of the existing structure may result in the identification of reserve capacities that may allow the MGDL to be raised. The live load specified in this Section for the system expansion is set higher than the MGDL to eliminate the possibility that the extensions, by design, have less capacity than the existing system. The economic impacts of the additional weight are considered minor.

No permit vehicle shall impose force effects higher than the transit vehicle.

### 3.05.6.3 Vertical Dynamic Load: IM

Vertical load due to the dynamic effects of the live load on continuous welded rail shall be computed in accordance with Equation 3.05.6.3-1.

\[
IM = \left( \frac{VCF}{K_c f_1} - 0.1 \right) LL \geq 0.20 LL \text{ kips} \quad \text{(Eq. 3.05.6.3-1)}
\]

in which:

\[
VCF = \text{vehicle crossing frequency defined as the vehicle speed divided by the span length (Hz)}
\]

\[
K_c = \begin{align*}
1.0 & \text{ for simple spans} \\
2.0 & \text{ for continuous spans}
\end{align*}
\]
\[ f_1 = \text{first mode flexural natural frequency of the structure (Hz)} \]

\[ LL = \text{axle live load in accordance with Section 3.05.6.2, excluding vertical dynamic load (IM) (kips)} \]

Vertical load due to the dynamic effects of the live load on switch frogs shall be computed in accordance with Equation 3.05.6.3-2.

\[ IM = 0.75LL \text{ kips} \quad \text{(Eq. 3.05.6.3-2)} \]

in which the variables are defined per Equation 3.05.6.3-1

This vertical load shall be applied at each axle location.

**Commentary:** The vertical dynamic load defined herein is developed from the following references:


The majority of research related to dynamic loads has concentrated on highway bridges, with relatively little information relating to pulse loading of structures with consistently repetitive axle spacing. A comparison of the previous AASHTO requirements utilized in the Phase I Design Criteria and the AREMA impact requirements for steel structures identifies significant
differences with the new criteria for long spans in the ranges proposed for the system extensions. The minimum vertical dynamic load has therefore been increased from 10 percent of the live load to 20 percent of the live load to provide more consistency between these sources.

ASCE 21-00 also provides a limit to the ratio of crossing frequency to natural frequency. The difference in the pulse loading dynamics between people mover vehicles (MetroMover four (4) car consist with eight (8) axles at 20') and a rapid transit vehicle (Metrorail four (4) car consist with 54', 21', 54', 21', 54', 21', 54' truck spacing) identifies greater structural sensitivity for the people mover vehicles than for rapid transit vehicles. The limit has not been incorporated herein for rapid transit vehicles.

Additional research and dynamic modeling is required to finalize these recommendations.

3.05.6.4 Horizontal Dynamic Load (Hunting Force): HF
Horizontal load due to the dynamic effects of the live load, defined as hunting force (HF), shall be computed as 10 percent of the live load (LL), excluding vertical dynamic load (IM). This force shall be equally distributed to the four axles of a vehicle, and shall be applied horizontally in the vertical plane containing each axle. This force shall be assumed to act in either direction transverse to the track through a point three feet six inches (3'-6") above the top of the lower rail.

3.05.6.5 Centrifugal Force: CF
Centrifugal force (CF) at each axle location shall be computed in accordance with Equation 3.05.6.5-1.
\[ CF = \frac{V^2 LL}{Rg} \text{ kips} \]  
(Eq. 3.05.6.5-1)

in which:

\[ V \] = maximum operating speed of the vehicle at the axle location (ft/sec)

\[ LL \] = axle live load in accordance with Section 3.05.6.2, excluding vertical dynamic load (IM) (kips)

\[ R \] = radius of curvature (ft)

\[ g \] = acceleration due to gravity (32.2 ft/sec²)

This radial force shall be applied horizontally in the vertical plane containing each axle, and shall act in a direction outward from the center of curvature, through a point four feet eight inches (4'-8") above the top of the lower rail. The effect of superelevation in reducing the overturning effect of centrifugal force on wheel loads may be considered.

Centrifugal force (CF) at all axle locations within a transition spiral shall be computed in accordance with Equation 3.05.6.5-1 using the minimum radius of curvature (R) defining the spiral.

3.05.6.6 Longitudinal Force: LF

Longitudinal force (LF) due to maximum service acceleration and deceleration shall be computed as 15 percent of the live load (LL), excluding vertical dynamic load allowance (IM). This force shall be applied as a load uniformly distributed over the length of a train in a horizontal plane four feet eight inches (4'-8") above the top of the lower rail. This force shall be assumed to act in either direction. For double track structures, combined forces due to either both trains accelerating or decelerating at the same time or one train accelerating and the other train decelerating shall be considered.
The longitudinal force (LF) due to emergency braking shall be computed as 25 percent of the live load (LL), excluding vertical dynamic load allowance (IM). This force shall be applied as a load uniformly distributed over the length of a train in a horizontal plane four feet eight inches (4'-8") above the top of the lower rail. This force shall be assumed to act in either direction. This force is used in the load combinations specified in Section 3.05.7 for Strength 6 Limit State only.

**Commentary:** The operational longitudinal force is calculated from the 3 mphps (4.4 ft/sec²) maximum service acceleration and deceleration (braking) specified in Volume VII, Sections 5.03.4 and 5.03.6.2, respectively, and includes the 10% tolerance increase. The emergency braking longitudinal force is calculated from the 4.5 mphps (6.6 ft/sec²) maximum brake rate specified in Volume VII, Section 5.03.6.4.

3.05.6.7 Live Load Surcharge: LS

Live load surcharges (LS) due to transit vehicle loading shall be computed as uniform surcharge, point, line and strip loads, as appropriate, in accordance with AASHTO LRFD Bridge Design Specifications Sections 3.11.6.1 through 3.11.6.3. AASHTO LRFD Bridge Design Specifications Sections 3.11.6.4 is not applicable to transit loading and shall not be used.

3.05.6.8 Pedestrian Live Load: PL

Pedestrian live load (PL) on all service and emergency walkways shall be a uniform load of 85 pounds per square foot of walkway area. Service and refuge areas under the station platform shall be designed for a uniform live load of 30 pounds per square foot of surface area. Pedestrian live load (PL)
shall be used for the design of walkway and refuge areas and their immediate supports only.

3.05.6.9 Derailment Load: DR, and Derailed Vehicle Dynamic Load: DI
Derailment load (DR) shall be produced by one or two married pairs of standard vehicles placed longitudinally parallel to the track. The force effects from the derailment load shall be calculated under the following load cases:

Load Case 1: The force effects from derailment (DR) shall be computed by placing the centerline of the vehicles four feet six inches (4’-6”) from the inside face of acoustical barrier/handrail. The vertical force for each vehicle shall be equal to the vehicle live load (LL). The derailed vehicle dynamic load (DI) shall include a vertical impact force equal to 100% of the vehicle live load applied through the wheels of the vehicle to the superstructure deck. The derailed vehicle dynamic load (DI) shall also include a transverse horizontal impact force equal to 50% of the vehicle live load (LL) and a longitudinal horizontal force equal to 10% of the vehicle live load (LL) applied along the top of the barrier/handrail over a distance of 15 feet.

**Commentary:** The derailment horizontal forces have been developed by approximate equivalence to a TL-5 bridge railing (refer to AASHTO LRFD Bridge Design Specifications Table 13.7.2-1) adjusted for vehicle weight, vehicle speed, and crash angle. These forces are also consistent with the recommendations of ACI 358.1R-92.
Load Case 2: If the acoustical barriers/handrails are designed to break away during a derailment the force effects from Load Case 1 may be reduced. The force effects from derailment (DR) shall be computed by placing the centerline of the vehicles two feet five and one half inches (2'-5½") from the inside face of acoustical barrier. The vertical force for each vehicle shall be equal to the vehicle live load (LL). The derailed vehicle dynamic load (DI) shall be equal to 75% of the vehicle live load applied vertically through the wheels of the vehicle to the superstructure deck. The force effects from Load Case 2 shall not be taken less than the force effects from the vertical components of DR and DI from Load Case 1 plus the load effects from the forces required to break away the barrier/handrail. The forces required to break away the barrier shall be calculated assuming reinforcing steel strain hardening, ultimate steel/aluminum tensile strength, maximum concrete strengths (at 100 year life), and other factors affecting the maximum force at which the acoustical barrier/handrail will fail.

Load Case 3: The force effects from derailment (DR) shall be computed by placing the centerline of the vehicles at the centerline of acoustical barrier/handrail. The vertical force for each vehicle shall be equal to LL. The derailed vehicle dynamic load (DI) need not be considered for this load case.

**Commentary:** This load case simulates a vehicle rolling over the edge of the guideway.
All areas of the superstructure deck between inside faces of the acoustical barriers shall be designed for a derailment load (DR) equal to the vehicle live load (LL) plus a derailed vehicle dynamic load (DI) equal to 100% of the vehicle live load (LL).

3.05.6.10 Wind on Structure: WS and Wind on Live Load: WL

Force effects from wind during system operation and during extreme wind events shall be considered. Wind on structure (WS) represents the force effects from extreme wind events and shall be calculated based on the basic wind speed and exposures prescribed by local building codes. The wind velocity for system operation conditions is 60 miles per hour. The adjustment of wind on structure (WS) for operational conditions is reflected in reduced load factors within the Limit States prescribed in Section 3.05.7.

Wind velocity pressures ($q_z$) shall be computed in accordance with Equation 3.05.6.10-1.

$$q_z = 0.00000256 \ K_z \ K_{zt} \ K_d \ K_r \ V^2 \ I \ ksf$$  \hspace{1cm} \text{(Eq. 3.05.6.10-1)}

in which:

$K_z$ = velocity pressure exposure coefficient
$K_{zt}$ = topographic factor
$K_d$ = wind directionality factor
$K_r$ = wind recurrence factor
$V$ = basic wind speed (miles per hour)
$I$ = importance factor
The velocity pressure exposure coefficient ($K_z$) shall be determined from ASCE 7-05 Table 6-3.

**Commentary:** Per local building codes (as of January 2006), the velocity pressure exposure coefficient shall not be less than prescribed for Exposure C.

The topographic factor ($K_{zt}$) shall be determined from ASCE 7-05 Figure 6-4.

The wind directionality factor ($K_d$) shall be taken as 1.00 to be consistent with the load factors provided in Section 3.05.7.

The wind recurrence factor, ($K_r$) shall be taken as 1.05 to reflect adjustments in wind velocity pressures from the 50-year recurrence identified by the local building code and the 100-year recurrence prescribed by the design life of the structure. Elements of the guideway requiring periodic replacement may be designed for a lower recurrence. Use of a lower year recurrence requires prior approval from MDT before incorporating into the design.

The basic wind speed ($V$) shall be determined in accordance with local building codes. Wind shall be assumed to come from any direction.

**Commentary:** Per local building codes (as of January 2006), the basic wind speed shall not be less than 146 miles per hour for a 50-year recurrence. The wind recurrence factor, $K_r = 1.05$, is based on a wind speed of 150 miles per hour in accordance with 100-year recurrence data for Miami-Dade County.

The importance factor, ($I$) shall be taken as 1.15.
Commentary: The importance of a recovery from an extreme wind event to provide service to the public as quickly as possible justifies the added safety provided by the importance factor.

Wind on structure (WS) shall be computed in accordance with Equation 3.05.6.10-2.

\[ WS = q_z GC A_{proj} \text{ kips} \]  
(Eq. 3.05.6.10-2)

in which:

- \( q_z \) = wind velocity pressure from Eq. 3.05.6.10-1 (ksf)
- \( GC \) = pressure coefficient, incorporating gust effect factor (G) and force coefficient (C)
- \( A_{proj} \) = projected area (ft²)

Pressure coefficients shall be developed for the aerial guideway and all attachments in accordance with ASCE 7-05. The following minimum pressure coefficients shall be used:

1. Main Wind-Force Resisting System (MWFRS) pressure coefficient for horizontal force shall be computed using the gust effect factor in accordance with ASCE 7-05, Section 6.5.8.1, and the force coefficient computed in accordance with Equation 3.05.6.10-3:

\[ C = 2.07 - 0.7 \frac{s}{h} \]  
(Eq. 3.05.6.10-3)

in which:
\[ S = \text{overall depth of superstructure including acoustical barriers and other deck appurtenances (ft)} \]

\[ h = \text{height to top of superstructure including acoustical barriers and other deck appurtenances (ft)} \]

Horizontal force shall be computed using a projected area, \( A_{proj} \), using the exposed area of the superstructure (including girder, acoustical barrier, and deck appurtenances) and substructure as projected into a vertical plane.

**Commentary:** The GC developed from Equation 3.05.6.10-3 is roughly equivalent to the use of ASCE 7-05 Figure 6-20 with an aspect ratio \( \geq 45 \).

2. MWFRS pressure coefficient for uplift force shall use a GC = 0.60 applied to the area of the superstructure (including girder, acoustical barrier, and deck appurtenances) and substructure as projected into a horizontal plane. The center of application of the wind uplift shall be the windward quarter point of the above described area.

The force effects due to wind on structure (WS) shall be computed assuming simultaneous application of horizontal and uplift forces. For various angles of wind direction, force effects in the transverse and longitudinal directions shall be calculated by resolving the wind velocity pressure into components normal to the surface upon which the wind is applied.

Wind on live load (WL) shall be computed in accordance with Equation 3.05.6.10-1 and 3.05.6.10-2 with the following modifications:
1. The wind recurrence factor, ($K_r$) shall be taken as 1.00.

2. The basic wind speed ($V$) shall be taken as 60 miles per hour.

3. The importance factor, ($I$) shall be taken as 1.00.

4. Pressure Coefficient (GC) shall be taken as 1.50.

**Commentary:** The GC of 1.50 corresponds closely to the pressure coefficient provided in the Phase I Design Criteria.

The force effects due to wind on live load (WL) shall be computed based on an exposed transit vehicle height of 11’-4” and width of 10’-6”. For various angles of wind direction, force effects in the transverse and longitudinal directions shall be calculated by resolving the wind velocity pressure into components normal to the surface upon which the wind is applied. The transverse force shall be applied to the rails and superstructure as loads concentrated at the axle locations, and in a plane six feet-four inches (6’-4”) above the top of the lower rail. The longitudinal force shall be applied to the rails and superstructure as a load uniformly distributed over the length of the train in a horizontal plane six feet-four inches (6’-4”) above the top of the lower rail. For the design of superstructure and substructure elements supporting two tracks, the wind on live load (WL) shall be increased by 30 percent when both tracks are loaded to account for the shielding effect of vehicle on vehicle as the two trains run alongside each other.
3.05.6.11 Thermal Effects: TU and TG
The force effects and movements resulting from temperature variations shall be calculated in accordance with the requirements of the FDOT Structures Design Guidelines Section 2.7.

3.05.6.12 Thermal Forces due to Rail Restraint: TTR, TLR, and RF
Rail-Structure interaction force effects due to the transverse and longitudinal restraint of thermal movement of continuous welded rail shall be considered in design of the aerial guideway. Force effects shall be computed for a temperature rise of 50 degrees F and fall of 70 degrees F assuming that the final welding of continuous welded rail is done when the rail segment has attained a mean temperature of approximately 100 degrees F. These force effects shall be applied in a horizontal plane at the top of the lower rail and shall act both transversely and longitudinally to the centerline of track as follows:

The force effects from transverse thermal rail restraint (TTR) per rail shall be assumed to act in either direction and shall be computed in accordance with Equation 3.05.6.12-1.

\[
TTR = \frac{A_r E_r \alpha_s (\Delta T)}{R} \quad \text{(kips/ft)} \quad \text{(Eq. 3.05.6.12-1)}
\]

in which:
\( A_r \) = area of rail (square inches)
\( E_r \) = modulus of elasticity of rail, taken as 29,000 ksi
\( \alpha_s \) = coefficient of thermal expansion of rail, taken as 0.0000065 inch/inch-degree F.
The force effects from longitudinal thermal rail restraint (TLR) per rail in sections of rail continuity shall be assumed to act in either direction and shall be computed in accordance with Equation 3.05.6.12-2.

\[
TLR = \frac{SL_f(\Delta L)}{S_f} \text{ kips} \quad \text{(Eq. 3.05.6.12-2)}
\]

in which:

- \( SL_f \) = fastener maximum longitudinal slip force, refer to Section 2.05.8 (kips)
- \( \Delta L \) = difference in the distance from the point of zero movement of the superstructure unit under consideration to the ahead and back superstructure expansion joints (ft)
- \( S_f \) = fastener spacing (ft)

TLR may be reduced to account for relief from the effects of horizontal shear restraint of bearings due to thermal movements of the girder.

The force effects from longitudinal thermal rail restraint (TLR) per rail at restrained rail termination points, e.g., crossovers, switches, rail terminations on structure. The restraining force per rail shall be computed in accordance with Equation 3.05.6.12-3

\[
TLR = A_r E_f \alpha_s (\Delta T) \text{ kips} \quad \text{(Eq. 3.05.6.12-3)}
\]

\( \Delta T \) = temperature change from temperature at rail welding (zero-stress condition) (degree F)

\( R \) = radius of curvature (ft)
in which the variables are defined per Equation 3.05.6.12-1.

The force effects from the release of continuous welded rail restraint associated with a single rail fracture (RF) shall be accommodated in the design of the aerial guideway. The rail-structure interaction system shall be designed to limit the rail gap at the fracture to 2 inches maximum unless otherwise approved by MDT.

3.05.6.13 Seismic Loads: EQ

Provisions for the design of the aerial guideway for seismic effects shall be in accordance with the FDOT Structures Design Guidelines, Section 2.3. The aerial guideway is considered non-exempt. The acceleration coefficient for Miami-Dade County shall be 2.5%.

3.05.7 LOAD FACTORS

Load factors for the aerial guideway design limit states shall be in accordance with Table 3.05.7-1. Values for \( \gamma_P \) and \( \gamma_{TG} \) shall be in accordance with the AASHTO LRFD Bridge Design Specifications, except \( \gamma_P \) shall be taken as 1.50 for component dead load (DC) for Strength 4 Limit State. \( \gamma_{PS} \) shall be taken as 1.00 for all applied loads, except for DW, where \( \gamma_{PS} \) shall be taken ranging from a minimum of 0.90 to a maximum of 1.10, whichever results in the more critical force effect.

**Commentary:** The load factor \( \gamma_{PS} \) is utilized for service load checks only, and allows for future adjustment of superimposed dead loads on the aerial guideway.

Load factors for construction loads shall be in accordance with AASHTO LRFD Bridge Design Specifications.
3.05.7.1 Variability of Loads

Transit vehicle live loads, buoyancy, wind loads and other variable loads shall be reduced or eliminated to create the maximum force effect on the structure. Transit vehicle live loads may be applied to all tracks affecting the structure. Loads for TV1 apply to the primary loading for the operational vehicle under consideration (including permit vehicle if applicable). Loads for TV2 apply to the second operational track and loads for TV3 apply to the third track with a stationary vehicle.

The full force effects from longitudinal force (LF) and horizontal dynamic load (HF) shall be applied to superstructure and substructure components of single track structures, dual track structures with one track loaded, and all dual track structures with both tracks loaded within 1,000 feet of ends of station platforms, including provisions for future extensions. For dual track structures, other than described above, LF and HF may be reduced by 25%.

3.05.7.2 Application of Dynamic Loads

Dynamic loads IM, HF, DI and IE shall be applied to the design of the superstructure, including the supporting columns. Dynamic loads shall not be applied to the design of abutments, retaining walls, and deep foundation elements below pile caps, with the exception that the portion above the point of fixity of deep foundation elements (e.g., piles, drilled shafts, etc.) which are rigidly connected to the structure above it.

3.05.7.3 Application of Horizontal Dynamic Load with Centrifugal Force

All superstructure and substructure elements shall be designed for centrifugal force (CF). When the Horizontal Dynamic Load (Hunting Force, HF), applied
in accordance with Section 3.05.7.2, acts simultaneously with CF, only the larger of the two forces needs to be considered.
### Limit States

<table>
<thead>
<tr>
<th>Loads</th>
<th>Sustained Loads</th>
<th>Transient Loads</th>
<th>Loads due to Volumetric Change</th>
<th>Exceptional Loads</th>
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<tr>
<td></td>
<td>Transit Vehicle</td>
<td>Natural Forces</td>
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</tr>
</tbody>
</table>

#### Table 3.05.7-1: Limit State Load Combinations and Load Factors

- **γ<sub>P</sub>**: Load factor for loads.
- **γ<sub>t</sub>**: Load factor for extreme events.
- **γ<sub>tg</sub>**: Load factor for exceptional loads.

<table>
<thead>
<tr>
<th>Limit States</th>
<th>Sustained Loads</th>
<th>Transient Loads</th>
<th>Loads due to Volumetric Change</th>
<th>Exceptional Loads</th>
</tr>
</thead>
<tbody>
<tr>
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<td>γ&lt;sub&gt;P&lt;/sub&gt;</td>
<td>γ&lt;sub&gt;t&lt;/sub&gt;</td>
<td>γ&lt;sub&gt;tg&lt;/sub&gt;</td>
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<td>Fatigue/Fracture 1</td>
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<td>1.00</td>
</tr>
</tbody>
</table>

**γ<sub>ps</sub>**: Load factor for service loads.
**γ<sub>ps</sub>**: Load factor for service loads.
3.05.8 STRUCTURAL DESIGN AND DETAIL REQUIREMENTS

The structural design of the aerial guideway shall be in compliance with the design and detail requirements of the FDOT Structures Design Guidelines, except as modified herein.

3.05.8.1 Uplift

Provision shall be made for adequate attachment of the superstructure to the substructure to resist uplift at any support. Attachment elements, which may be anchor bolts or other mechanical hold-down devices, shall be designed in accordance with the AASHTO LRFD Design Specifications. No uplift at the attachment of the superstructure to the substructure shall be allowed under Service 1 Limit State. Uplift anchorage and bearing details shall be designed to electrically isolate the substructure for stray currents from the superstructure in accordance with Section 3.03.5.

No uplift in concrete deep foundation elements shall be allowed under sustained loads included in Service Limit State 1 without live load. Where documentation is provided demonstrating that this uplift restriction cannot be eliminated in a cost-effective manner, special designs will be considered on a case-by-case basis, and requires prior approval from MDT before incorporating into the design.

3.05.8.2 Vibration and Deflection Control

To limit vibration amplification due to the dynamic interaction between the superstructure and the transit vehicle, the computed first mode natural frequency of flexural vibration of the superstructure shall not be less than 2.50 cycles per second.

To ensure rider comfort, the deflection of longitudinal girders under Service Limit State 1 with 90 percent of live load (LL) plus vertical dynamic load (IM)
shall not exceed 1/1000 of the span. The differential deflection of the slab immediately below the centerlines of the two rails of the same track, due to girder and slab deformations, shall not exceed 1/5000 of the span length for Service Limit State 1 modified for a reduced wind speed of 27 mph.

The long-term differential settlement between two adjacent piers supporting the aerial guideway superstructure shall not exceed 1/2400 of the sum of the lengths of any two adjacent affected spans; however, long-term differential settlement between piers supporting guideway girders adjacent to station platforms (including future extensions) shall not exceed 3/8 inch.

3.05.8.3 Fatigue Design

In addition to stress and capacity requirements, the design of the aerial guideway structure shall consider the effects of accumulated stress range cycles due to the passage of loads identified the Fatigue/Fracture 1 Limit State. The number of vehicle consists to be considered in this Limit State for the design life of the structure shall be based on the Load Occurrence Spectrums for single track guideway in Table 3.05.8.3-1 and for dual track guideway identified in Table 3.05.8.3-2.
The table below provides the load occurrence spectrum for single and dual track guideways, detailing the vehicle weight and passenger load conditions.

### Single Track Guideway

<table>
<thead>
<tr>
<th>Loading</th>
<th>Vehicle Weight (kips)</th>
<th>6-Car Consist</th>
<th>2-Car Consist</th>
<th>Total Vehicle Consists</th>
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<tr>
<td>Empty Vehicle</td>
<td>82.5</td>
<td>-</td>
<td>-</td>
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<tr>
<td>Passengers Seated</td>
<td>94.8</td>
<td>4,400,000</td>
<td>1,000,000</td>
<td>5,400,000</td>
</tr>
<tr>
<td>½ Standing Passengers</td>
<td>108.0</td>
<td>2,700,000</td>
<td>600,000</td>
<td>3,000,000</td>
</tr>
<tr>
<td>Full Live Load (LL)</td>
<td>120.0</td>
<td>300,000</td>
<td>0</td>
<td>300,000</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td></td>
<td></td>
<td><strong>9,000,000</strong></td>
</tr>
</tbody>
</table>

Table 3.05.8.31: Load Occurrence Spectrum – Single Track Guideway

### Dual Track Guideway

<table>
<thead>
<tr>
<th>Loading</th>
<th>Vehicle Weight (kips)</th>
<th>One Track Loaded (TV1 or TV 2) each Track</th>
<th>2 Tracks Loaded (TV1 &amp; TV 2)</th>
<th>Total Vehicle Consists</th>
</tr>
</thead>
<tbody>
<tr>
<td>Empty Vehicle</td>
<td>82.5</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Passengers Seated</td>
<td>94.8</td>
<td>3,300,000</td>
<td>750,000</td>
<td>10,800,000</td>
</tr>
<tr>
<td>½ Standing Passengers</td>
<td>108.0</td>
<td>2,000,000</td>
<td>450,000</td>
<td>6,600,000</td>
</tr>
<tr>
<td>Full Live Load (LL)</td>
<td>120.0</td>
<td>200,000</td>
<td>-</td>
<td>600,000</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td></td>
<td></td>
<td><strong>18,000,000</strong></td>
</tr>
</tbody>
</table>

Table 3.05.8.3-2: Load Occurrence Spectrum – Dual Track Guideway
The number of accumulated stress range cycles shall be developed considering all variations of stress for components of both superstructure and substructure occurring due to the passage of the vehicle consists identified in the Load Occurrence Spectrum.

**Commentary:** The number of fatigue cycles experienced by the guideway is dependent on such factors as span length and structural continuity at supports. A 40 foot simple span will experience more than six times the number of maximum stress range cycles as a 130 foot simple span under the same 6-car consist loading. The effects of variable amplitude stress ranges shall be evaluated using Miner’s Rule of Cumulative Damage, or other methods acceptable to MDT.

3.05.8.4 Distribution of Loads

The distribution of loads to the structural elements in the aerial guideway shall be in accordance with acceptable methods of analysis as described in the AASHTO LRFD Bridge Design Specifications. Reduction in loads due to the presence of multiple vehicles shall not be taken into account, except as indicated in these criteria.

3.05.8.5 Vehicle Wheel Contact Area

Under service condition, each wheel load shall be dispersed through the running rail to produce an effective contact area of six inches (measured parallel to rail) by 12 inches measured at the bottom of the elastomeric bearing pad under rail fastener.

Under derailment condition, each derailed wheel in direct contact with plinth or deck slab shall be assumed to make a groove 1/4 inch to 1/2 inch deep with a wheel to concrete surface area two inches wide by eight inches long.
3.05.8.6 Access and Maintenance for Box Sections

Provide access and maintenance design and details in accordance with the requirements of the FDOT Structures Design Guidelines and FDOT Design Standard Index 21240 for all steel box and post-tensioned concrete box sections, with the single exception that the provisions shall include electrical outlets with two duplex receptacles spaced at no more than 50 feet, and that provisions for surface mounted incandescent light fixtures are not required.
3.06  CONCRETE STRUCTURES

3.06.1  GENERAL

All concrete members shall be designed using the Load and Resistance Factor Design Method in accordance with these criteria, the FDOT Structures Design Guidelines and the AASHTO LRFD Bridge Design Specifications in that order of precedence.

Finish and appearance of concrete members shall comply with the requirements of relevant provisions of Section 1.01.

3.06.2  MODULUS OF ELASTICITY OF CONCRETE

The modulus of elasticity of concrete shall be computed in accordance with AASHTO LRFD Bridge Design Specifications Equation 5.4.2.4-1. The correction factor \( K_1 \) shall be taken as 0.90 for concrete using Oolitic limestone aggregates. The concrete compressive strength \( f'c \) shall be taken as the specified concrete compressive strength for all stress check conditions specified herein. Higher concrete strengths corresponding to actual strengths at time of load application may be used only for vibration and deflection control in Section 3.05.7.2.

3.06.3  CRITICAL TENSION LIMITS

Tensile stresses in prestressed concrete superstructure and substructure elements shall be limited to the following:

1. No tension shall be permitted for elements subjected to Service 3 Limit State with 90 percent of live load (LL) plus vertical dynamic load (IM).
2. Tension for non-segmental construction under Service 3 Limit State with 100 percent of live load (LL) plus vertical dynamic load (IM) shall not exceed $2.25 \sqrt{f'^c}$.

3. Stresses for segmental construction shall be checked at joints to ensure a minimum flexural service compression of 50 psi under Service 3 Limit State with 100 percent of live load (LL) plus vertical dynamic load (IM).

4. Tension under Service 6 Limit State shall not exceed $7.50 \sqrt{f'^c}$. Transit vehicle loads other than the derailed vehicle shall be applied with 100 percent of live load (LL) plus vertical dynamic load (IM), as appropriate.

5. For post-tensioned construction, the principal tensile stress resulting from the long-term residual axial stress and maximum shear and/or maximum shear combined with shear from torsion stress at the neutral axis of the critical web under Service 3 Limit State shall not exceed $3.00 \sqrt{f'^c}$.

6. For the design of members supporting multiple tracks, the specified service tension limits shall not be exceeded when either or any combination of tracks are loaded.
3.07 STEEL STRUCTURES

3.07.1 GENERAL

All steel members shall be designed using the Load and Resistance Factor Design Method in accordance with these criteria and the *AASHTO LRFD Bridge Design Specifications* in that order of precedence.

Finish and appearance of steel members shall comply with the requirements of relevant provisions of Section 1.01.