FLORIDA DEPARTMENT OF TRANSPORTATION

FDOT MODIFICATIONS TO STANDARD SPECIFICATIONS FOR STRUCTURAL SUPPORTS FOR HIGHWAY SIGNS, LUMINAIRES AND TRAFFIC SIGNALS (LTS-6)

FDOT STRUCTURES MANUAL
VOLUME 3
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1 INTRODUCTION

1.1 Scope

Add the following:

Conform to the date specific AASHTO Publications listed in ***Structures Manual Introduction*** I.6 References.

C 1.1

Add the following:

Structures Manual Introduction I.6 is updated annually to reflect the specific specifications editions and interims adopted by the FDOT.

2 GENERAL FEATURES OF DESIGN

2.1 Scope

Add the following:

See Chapters 2, 7 and 29 of the FDOT ***Plans Preparation Manual***, Volume 1 regarding the use of FDOT ***Design Standards*** and other plans preparation requirements.

C 2.1

Add the following:

The FDOT ***Plans Preparation Manual*** contains additional FDOT requirements for sign, signal and lighting structures. The FDOT ***Design Standards*** contains drawings for all typical sign, signal and lighting structures.

2.4 Functional Requirements

2.4.2 Structural Supports for Signs and Traffic Signals

2.4.2.2 Size, Height and Location of Signs

Add the following:

Span type overhead sign structures in urban locations shall be designed either for the actual signs shown on the signing plans or for a minimum sign area of 120 sq. ft. (12 ft. W x 10 ft. H) per lane, whichever is the greater. The minimum sign area applies to lanes without signs and lanes with sign sizes smaller than the minimum. If the signing plans require signs for only one traffic direction, the minimum sign area per lane requirement applies to the traffic lanes in this direction only.

Cantilever type overhead sign structures in urban locations shall be designed either for the actual signs shown on the signing plans or for a minimum sign area of 80 sq.

C 2.4.2.2

Add the following:

Minimum sign areas provide a reasonable allowance for future sign panel installations without the need for a new support structure.

Minimum sign areas for overhead variable message sign supports are normally not required.

See the FDOT ***PPM***, Volume 1, Introduction for a link to the Urban Area Boundary Maps. See ***PPM***, Volume 1 for cantilever and span overhead sign support location criteria.

ft. (8 ft. W x 10 ft. H) located at the end of the cantilever, whichever provides the more stringent load or stress at the location under consideration.

Figures 1 and 2 show how to apply the above minimum sign areas for span type overhead sign structures in urban locations.

Overhead signs in rural locations should be designed for the actual sign shown on the signing plans.

Figure 1 Example: Actual Signs

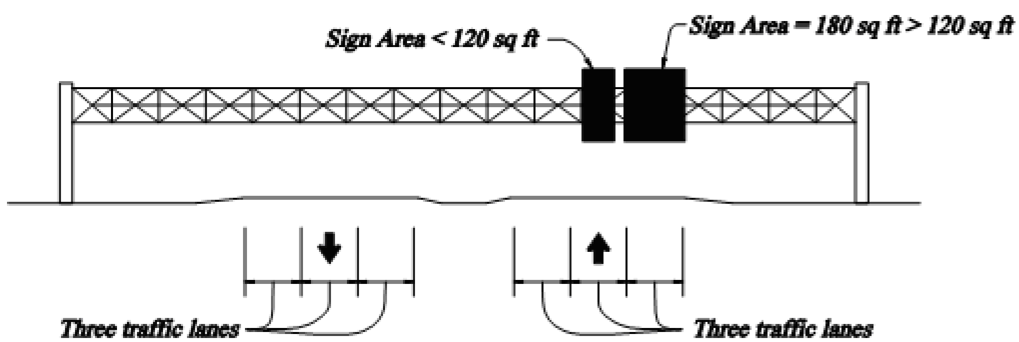
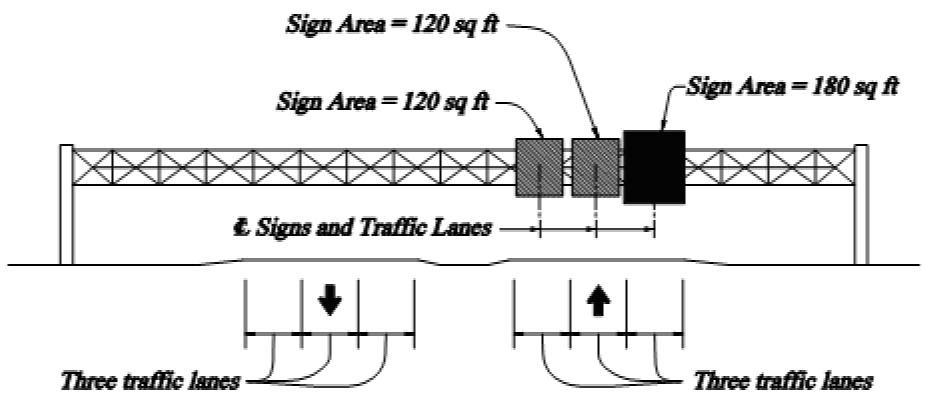


Figure 2 Example: Signs Used in Design



2.4.2.4 Variable Message Signs

Add the following:

For all overhead Variable Message Sign (VMS) structures, the horizontal member shall consist of a truss with a minimum of two chords with a minimum center-to-center distance between the chords of 3'-0". See FDOT section 11.8 for VMS maximum span-to-depth ratios.

FDOT vertical clearance requirements for VMS structures are found in ***PPM***, Volume 1, Chapter 2.

C 2.4.2.4

Add the following:

The minimum requirements given provide additional measures to limit the possibility of galloping.

Since cantilever overhead Variable Message Sign (VMS) structures are more susceptible to fatigue than span overhead VMS structures, span structures should be used whenever possible.

In Florida, overhead VMS structures are typically referred to as Dynamic Message Sign (DMS) structures.

2.4.2.5 Horizontal Span and Cantilever Limits

New Section, add the following:

See ***PPM***, Volume 1, Section 29.1 for sign and signal structure limits.

3 LOADS

3.8 Wind Load

Delete the last paragraph and add the following:

The use of Appendix C is only permitted for the evaluation of existing structures.

C 3.8

For existing supports, FDOT ***PPM***, Volume 1, Section 25.4.26 defines when structural evaluation is necessary and lists FDOT Design Exception and Variation requirements.

3.8.2 Basic Wind Speed

Delete the entire paragraph including Figures 3.8.3-2, and 3.8.3-4 and add the following:

The wind loads shall be based on the wind speeds (mph) shown in FDOT ***SDG*** *Table 2.4.1-2*

C 3.8.2

Add the following:

FDOT ***SDG*** *Table 2.4.1-2* was derived from the ASCE 7-05 wind speed map.

To simplify the design process, FDOT has designated one wind speed per county.

3.8.3 Wind Importance Factor I_r

Add the following Wind Importance Factor to Table 3.8.3-1:

Recurrence Interval Years	V = 85-100 mph	V > 100 mph	Alaska
1.5	0.45	0.2	---

Delete Table 3.8.3-2 and add the following FDOT Table 3.8.3-2:

FDOT Table 3.8.3-2 Minimum Design Life

Design Life	Structure Type
50-year	Overhead sign structures
	Luminaire support structures >50' in height.
	Mast Arms
	Monotubes
	Steel Strain Poles
	ITS Camera Poles >50' in height
25-year	Luminaire supports and other structures ≤ 50' in height.
	Concrete Strain Poles
10-year	Roadside sign structures
1.5-year	Temporary construction signs

A 1.5-year design life (I_r = 0.2) for temporary construction signs shall only be used with a 150 mph design wind speed.

C 3.8.3

Add the following:

A 1.5-year design life has been added for temporary construction signs. The importance factor is calculated based on "Wind Speed for Design of Temporary Structures" by D.W. Boggs and J.A. Peterka, Structures Congress, 1992, Compact Papers, ASCE, 1992.

Florida has traditionally designed Luminaire support structures, 50 feet in height and less, and strain poles for a 25 year design life.

Concrete strain poles are designed for zero tension stress, therefore a twenty-five year design life is appropriate.

3.8.6 Drag Coefficients C_d (Rev. 01/16)

Add the following to Table 3.8.6-1:

Traffic Signals - no ability to swing	1.2
Traffic Signals - installed with the ability to swing	0.7
Solar Panels - installed with a tilt angle between 15 and 30 degrees	2.1 (positive) 1.8 (negative)

C 3.8.6

Add the following to note 2 at the bottom of Table 3.8.6-1:

A drag coefficient for traffic signal installed with the ability to swing has been established through research (Cook 2007). On span wire systems where signal and signs are allowed to swing, varying C_d as a function of swing angle is allowed (Hoit and Cook 1997).

The coefficients given for solar panels are approximately the same as the ones given in ASCE 7-10, Figure 27.4-4 for inclined monosloped roofs. See simplified illustration in FDOT Figure 3.8.6.

FDOT Figure 3.8.6-1 Drag Coefficients for Solar Panels

3.8.7 Lift Coefficient for Traffic Signals C_l

New Section, add the following:

For traffic signals installed with the ability to swing on span wire systems under full design wind speed (Group II loading), use a coefficient of lift C_l equal to 0.4. To compute the lift pressure, use Eq. 3.8.3-1 substituting C_l for C_d. Using a reduced signal area based on the swing angle, compute the lift force and apply in a vertical direction opposite dead load.

C 3.8.7

Add the following:

A lift coefficient of 0.4 on traffic signals installed on span wire systems has been established through research (Cook 2007). On span wire systems where signal and signs are allowed to swing, varying C_l as a function of swing angle is allowed (Hoit and Cook 1997).

3.9 Design Wind Loads On Structures

3.9.1 Load Application

Add the following:

Use the following areas for traffic signals:

Item	Projected Area
12" Signal	1.36 sf
8" Signal	0.70 sf
3 Section Backplate	5.67 sf
4 Section Backplate	6.83 sf
5 Section Backplate	8.00 sf

When the full design wind speed is used for Group II loading on span wire systems, use a reduced signal and sign area based on the following swing angles:

Wind Speed	Swing Angle
110 mph	45 degrees
130 mph	55 degrees
150 mph	60 degrees

C 3.9.1

Add the following:

Swing angles for traffic signals and signs installed on span wire systems have been established through research (Cook 2007).

Areas given are for standard signals in Florida.

3.9.3 Design Loads for Vertical Supports

Add the following:

When 3 or 4 span wire pole structures are connected, analyze the system with wind directions of 0, 45, and 90 degrees. If other angles are used, document the angles in the analysis report.

C 3.9.3

Add the following:

More refined analysis is typically not required due to the number of approximate assumptions made in the analysis. Other angles may be analyzed and substituted if program results are not consistent at the specified angles.

3.10 References

Add the following:

Cook, R.A. (2007). ***Development of Hurricane Resistant Cable Supported Traffic Signals*** (FDOT Report# BD545 RPWO #57). Gainesville, Florida: University of Florida.

Hoit, M.I., Cook, R.A. (1997). ***Computer Aided Design Program for Signal Pole and Span Wire Assemblies With Two Point Connection System*** (FDOT Report# 0510653). Gainesville, Florida: University of Florida.

5 STEEL DESIGN

5.5 Material - Structural Steel

Add the following:

Do not specify ASTM A588 (rustic, Corten, “self-oxidizing", or "self-weathering") steel in sign, signal, or lighting structures.

C 5.4

Add the following:

In some environmental conditions in Florida, A588 steel has deteriorated significantly faster than expected.

5.13 Cables And Connections

Add the following:

Use the cable breaking strength values specified in FDOT ***Specifications*** Section 634.

C 5.13

Add the following

Cables used in the construction of span-wire pole structures are listed in FDOT ***Specifications*** Section 634.

5.14 Details of Design

5.14.3 Transverse Plate Thickness

Add the following:

The minimum base plate thickness shall be 2½ inches for mast arm signal structures and steel strain poles, and 3 inches for high mast light poles.

C 5.14.3

Add the following:

Research has proven full-penetration groove welds combined with thicker base plates increases the pole-to-base-plate connection fatigue strength.

5.15 Welded Connections

C 5.15
Add the following:
Section 5.15 is referenced as a requirement in FDOT ***Specifications*** *Section 460-6.4.*

5.15.1 Tube-to-Tube Splice Circumferential Welds (Rev. 01/16)

Add the following:

On steel sign and signal support structures, no circumferential welds are permitted on the uprights, arms or chords with the following exceptions:

- The upright to base plate weld
- The flange plate connection weld on tubular truss chords
- Mitered arm-to-upright angle welds on monotubes
- Uprights with lengths greater than available mill lengths.

C 5.15.1
Add the following:
The Department's intent is to avoid any unnecessary welds on sign, signal or lighting structures.

Typical mill lengths are 35 feet and greater.

5.15.3 Tube-to-Transverse Plate Connection Welds

Add the following:

For base plate connections without stiffeners on 50 year recurrence interval structures, only use full-penetration groove welds.

C 5.15.3
Add the following:
Research has proven full-penetration groove welds combined with thicker base plates increases the pole-to-base-plate connection fatigue strength.

5.16 Bolted Connections (Rev. 01/16)

Add the following:

Design all pole to arm connections on Mast Arm structures as "through bolted" using a minimum of six bolts. Do not use hardened steel washers between the end plate of a Mast Arm and the mounting plate of the pole.

C 5.16
Add the following:
Tapped connections are not permitted. Through bolted connections provide fully tensioned A325 bolts.

5.17 Anchor Bolt Connections

Add the following:

All sign, signal, and lighting structures designed for a minimum service life of 50 years (wind speed based on a 50-year mean recurrence interval) shall use a minimum of eight, Grade 55, ASTM F1554 anchor bolts at the pole to foundation connection, with the exception of Mast Arm signal structures where the minimum is six anchor bolts.

C 5.17

Add the following:

A minimum of eight anchor bolts provides redundancy and better distribution of forces through the base plate.

5.17.1 Anchor Bolt Types

Delete anchor bolts types listed in the second and third bullet and add the following:

Both Adhesive anchors and threaded post-tensioning bars are not permitted.

C 5.17.1

Add the following:

FDOT only allows straight headed anchor bolts.

Adhesive anchor and threaded post-tensioning bars have undesirable creep and non-ductile behavior respectively.

5.17.2 Anchor Bolt Materials

Add the following:

Only use ASTM F 1554 anchor bolts with 55 ksi yield strength.

C 5.17.2

Add the following:

ASTM F 1554 Grade 55 anchor bolts provide sufficient ductility after yield to engage all the anchor bolts on the tension side of the base plate.

5.17.2.4 Anchor Bolt Holes in Base Plate

Delete the first bullet in the first paragraph.

C 5.17.2.4

Oversized anchor bolt holes are allowed in Florida.

5.17.3 Design Basis

5.17.3.1 Double-Nut Anchor Bolt Connections

Add the following:

Use double-nut moment joints in all mast arm signal structures, steel strain poles, high mast light poles and overhead sign structures.

Replace the 2nd paragraph with the following:

In cantilever support structures, bending of the anchor bolt from shear and torsion shall be considered according to Article 5.17.4.3 unless a structural grout pad is provided. In non-cantilevered support structures, if the clear distance between the bottom of the bottom leveling nut and the top of concrete is less than the nominal anchor bolt diameter, bending of the anchor bolt from shear and torsion may be ignored. If the clear distance exceeds one bolt diameter, bending in the anchor bolt shall be considered according to Article 5.17.4.3.

C 5.17.3.1

Add the following:

A structural grout pad significantly contributes to the design load carrying capacity of anchor bolts in cantilever structures.

5.17.3.3 Use of Grout (Rev. 01/16)

Add the following:

A structural grout pad is required under the base plates in double-nut moment joints of mast arm signal structures and cantilever overhead sign structures.

Grout pads are not required under the base plates in double-nut moment joints of span overhead sign structures, high mast light poles, and steel strain poles.

C 5.17.3.3

Add the following:

When significant torsion is transmitted from the base plate to the anchor bolt group, a structural grout pad permits the anchors to develop their full shear strength, Cook et al. (2013).

Inspections have shown that a poorly functioning grout pad is worse than no grout pad at all. For poles without a grout pad beneath the base plate, the double-nut moment joint requires adequate tensioning of the anchor bolts. It is critical that the nuts beneath the base plate, typically referred to as leveling nuts, are firmly tightened and locked to prevent loosening. This locking mechanism is accomplished through the turn of the nut method specified in FDOT ***Specifications*** Section 649 or a properly placed grout pad.

5.17.4 Anchor Bolt Design

5.17.4.3 Bending Stress in Anchor Bolts

Replace the first paragraph with the following:

In cantilevered support structures, bending stresses in the anchor bolts shall be considered unless a structural grout pad is provided. In non-cantilevered support structures, when the clearance between the bottom of the leveling nuts and the top of the concrete foundation exceeds one bolt diameter, bending stresses in the anchor bolts should be considered.

C 5.17.4.3

Replace the first paragraph with the following:

Bending stresses in individual bolts can be ignored in the following cases:

- in non-cantilevered support structures, if the standoff distance between the top of the foundation and bottom of the leveling nut is less than one bolt diameter.
- in cantilevered support structures, if a structural grout pad is provided.

For larger standoff distances, the following beam model should be used. (See LTS.)

5.19 References

Add the following:

Cook, R. A., Prevatt, D. O., and McBride, K. E. 2013. *Steel Shear Strength of Anchors with Stand-Off Base Plates*. Florida Department of Transportation Research Report BDK75-49, Tallahassee, FL

6 ALUMINUM DESIGN

6.1 Scope

Add the following:

Do not specify aluminum overhead sign structure supports with the exception of the vertical sign panel hangers, which may be aluminum or steel.

C 6.1

Add the following:

Aluminum overhead sign structures have been prone to unacceptable levels of vibration and fatigue cracking.

7 PRESTRESSED CONCRETE DESIGN

7.5 Design

7.5.1 Method of Design

Add the following:

For Standard Prestressed Concrete Pole Design, see ***Instructions for Design Standards*** Index 17725, for the Service Moment Capacity and Ultimate Moment Capacity. An increased percentage of Allowable Stress for Group II loading (Table 3.4-1) is not applicable for Prestressed Concrete Poles, since Group II loading is an ultimate moment capacity calculation.

C 7.5.1

Add the following:

FDOT uses Standard Prestressed Concrete Poles in accordance with Index 17725 and ***Specifications*** Section 641. After analysis of the proposed span-wire pole structure, the Designer selects the appropriate pole using the design moment values given in the ***Instructions for Design Standards*** for Index 17725.

7.5.2 Concrete Strength

Replace this section with the following:

The minimum compressive concrete strength shall be 6 ksi.

C 7.5.2

Add the following:

FDOT uses Class V Special, 6 ksi or Class VI 8.5 ksi concrete in accordance with ***Specifications*** Section 346.

7.10 Durability

7.10.2 Concrete Cover

Replace this section with the following:

The minimum clear concrete cover for all prestressed and non-prestressed poles is 1 inch.

C 7.10.2

Add the following:

FDOT requires a minimum 1 inch cover on all concrete poles in all environments.

10 SERVICEABILITY REQUIREMENTS

10.5 Camber (Rev. 01/16)

Replace this section with the following:

Provide a design camber equal to 2.5 times the dead load deflection for overhead sign structures. For span overhead sign structures, arch the horizontal member upwards and for cantilever overhead sign structures rake the vertical support backwards. For mast arm signal structures, provide a two degree upward angle at the arm/upright connection.

C 10.5

Add the following:

Design camber = Permanent camber + dead load deflection. Permanent camber equal to 1.5 times the dead load deflection provides for a better appearance than the relatively small L/1000 given in AASHTO. For mast arms, a two degree upward angle at the arm/upright connection is standard industry practice.

11 FATIGUE DESIGN

11.6 Fatigue Importance Factors

Add the following:

When evaluating galloping, use Fatigue Category II for all flat panel sign, traffic signal, and lighting support structures meeting the limits in 2.4.2.5 and designed in accordance with the current LTS specifications. Use Fatigue Category I for all other sign, traffic signal, and lighting support structure designs including all VMS support structures.

C 11.6

Add the following:

There have been no reports of fatigue damage to sign, signal and lighting structures built using FDOT ***Design Standards***.

11.7 Fatigue Design Loads

11.7.1 Galloping

Replace the 2nd, 3rd and 4th paragraphs with the following:

Vibration Mitigation devices are not allowed in lieu of designing for galloping.

Exclude galloping loads for the fatigue design of overhead cantilevered sign and VMS support structures with three or four chord horizontal trusses with bolted web to chord connections.

C 11.7.1

Add the following:

Vibration mitigation devices are seldom necessary and installed only after excessive vibration has been observed and the device is approved by the Department.

Cantilevered sign support structures with horizontal three or four chord trusses have never been reported to vibrate from vortex shedding or galloping. (ref. FHWA Guidelines for the Installation, Inspection, Maintenance and Repair of Structural Supports for Highway Signs, Luminaries, and Traffic Signals)

11.8 Deflection

Add the following:

In addition, VMS structures shall also meet the following maximum span-to-depth ratios:

VMS Structure Type	Max. Span-to-Depth Ratio
Overhead Span Structure	25
Overhead Cantilever Structure	9

C 11.8

Add the following:

The minimum requirements given provide additional measures to limit the possibility of galloping

13 FOUNDATION DESIGN

13.6 Drilled Shafts

Add the following:

Drilled shafts are the standard foundation type on high mast light poles, overhead signs, mast arms and steel strain poles.

C 13.6

Add the following:

For standard drilled shaft details, see ***Design Standards*** Indexes 11320, 17502, 17723 and 17745 for span overhead sign structures, high mast light poles, steel strain poles, and mast arms respectively.

13.6.1 Geotechnical Design

13.6.1.1 Embedment (Rev. 01/16)

Add the following:

Use a safety factor against overturning of 2 when using the Broms method.

For torsion resistance in drilled shafts supporting Mast Arm signal and cantilever overhead sign structures, use the following equations:

$$T_u \leq \frac{T_n}{SF_{tor}}$$

Where

$$T_n = \pi D L F_s \left(\frac{D}{2}\right)$$

$$F_s = \sigma_v \omega_{fdot}$$

$$\sigma_v = \gamma_{soil}\left(\frac{L}{2}\right)$$

T_u = Torsion force on the drilled shaft

T_n = Nominal torsion resistance of the drilled shaft

SF_{tor} = Safety Factor against torsion
= 1.0 for Mast Arm signal structures
= 1.3 for overhead cantilever sign structures

D = diameter of the drilled shaft

L = length of the drilled shaft

F_s = unit skin friction

σ_v = effective vertical stress at mid-layer

ω_{fdot} = load transfer ratio where the allowable shaft rotation may exceed 10 degrees
= 1.5 for granular soils where uncorrected SPT N-values are 15 or greater
= $1.5\left(\frac{N-value}{15}\right)$ for uncorrected N-values greater than or equal to 5 and less than 15.

γ_{soil} = unit weight of soil

C 13.6.1.1

Add the following:

FDOT experience has established a safety factor of 2 produces conservative designs.

The torsion resistance equation is based on the theory for the Beta Method (O'Neill and Reese, 1999). The torsional resistance from the bottom face of the shaft is omitted to increase the conservatism in this approximate calculation. A single ω_{fdot} factor of 1.5 is used to adjust for the concurrent overturning and torsional forces and to compare with past FDOT practice. Since the consequence of a torsion soil-structure failure is usually small, some rotation may occur from the design wind.

Since cantilever overhead sign structures can have significantly more torsion than a Mast Arm, a higher safety factor of 1.3 is appropriate.

For soils with SPT N-values less than 5, consult the Geotechnical Engineer for additional recommendations.

13.6.2 Structural Design

Add the following:

Longitudinally reinforce drilled shaft foundations with a minimum of 1% steel. At a minimum, place #5 stirrups at 4 inch spacing in the top two feet of shaft. In cantilever structures, design for shear resulting from the torsion loading on the anchor bolt group.

C 13.6.2

Add the following:

Using 1% steel is conservative for flexural design in most cases. Additional stirrups in the top of the shaft provides resistance against shear failure in the top of the shaft. Due to torsion, additional stirrups may be required in cantilever structures.

13.6.2.1 Details (Rev. 01/16)

Replace the second sentence with the following:

A minimum concrete cover of six inches over steel reinforcement is required.

Add the following:

The minimum design diameter for drilled shafts is 3 feet and the maximum design diameter is 6 feet. A minimum reinforcement clear spacing of six inches is required for proper concrete consolidation. The top five feet of stirrups in drilled shafts for sign, signal and lighting structures are exempt from this spacing requirement.

C 13.6.2.1

Add the following:

FDOT requires six inches of cover to ensure durability in drilled shafts.

The concrete in drilled shafts with design diameters greater than 6 feet is considered mass concrete, therefore shafts of this size should be avoided.

Concrete consolidation below the anchor bolts becomes more difficult with reinforcement clear spacing less than six inches. Larger shaft diameters should be considered to increase reinforcement spacing.

Modification for Non-Conventional Projects:
Delete FDOT 13.6.2.1 and insert the following: *Replace the second sentence with the following:* A minimum concrete cover of six inches over steel reinforcement is required. *Add the following:* A mass concrete placement plan is required for drilled shafts with design diameters greater than 6 feet. A minimum reinforcement clear spacing of six inches is required for proper concrete consolidation. The top five feet of stirrups in drilled shafts for sign, signal and lighting structures are exempt from this spacing requirement.

13.10 Embedment of Lightly Loaded Small Poles and Posts

Add the following:

When using the Broms method for ground sign foundation design, use a safety factor against overturning of 1.3. When using the Broms method for direct burial concrete pole foundation design, use a safety factor against overturning of 1.5.

13.11 References

Add the following:

Cook, R.A. (2007). ***Anchor Embedment Requirements for Signal/Sign Structures*** (FDOT Report# BD545 RPWO #54). Gainesville, Florida: University of Florida.

APPENDIX C

C.1 Alternate Method (Rev. 01/16)

Add the following:

When using Appendix C:

- an allowable overstress factor (LTS 3.4) of 1.4 for Group II loading is allowed.
- FDOT minimum sign areas (FDOT 2.4.2.2) are not required.
- fatigue evaluation (LTS Section 11) is not required.
- foundation evaluation (LTS Section 13), structural and geotechnical, is not required.

In addition, if any of the following details exist, a Design Variation/Exception is not required:

- mast arm to upright connections with 4 bolts (FDOT 5.16)
- tapped mast arm connections (FDOT 5.16)
- fillet welded tube-to-transverse plate connections (FDOT 5.15.3)
- mast arm upright anchorages with 4 bolts (FDOT 5.17)
- transverse plate thickness (FDOT 5.14.3)
  - mast arm horizontal and upright 1.5 inches and greater
  - high mast light pole and steel strain pole 2.0 inches and greater

C C.1

Add the following:

By allowing an overstress factor of 1.4, consistent with previous editions of LTS, properly designed existing structures will be allowed to remain in place in accordance with the ***PPM***.

All items listed above should be checked in situations where there is evidence of distress or instability, or where the Engineer has reason to believe the structural capacity is in doubt.

C.2 Wind Load

Delete the 2nd and 3rd sentence and add the following:

The design wind pressures shall be computed using the wind pressure formula, Eq. C.3-1, with the appropriate wind speed shown in FDOT Table C.2-1, Wind Speed by County.

C C.2

Add the following:

To simplify the design process, FDOT has designated one wind speed per county.

FDOT Table C.2-1 Wind Speed by County

County (Dist)	10 year	25 year	50 year	County (Dist)	10 year	25 year	50 year
Alachua (2)	60	80	90	Lee (1)	80	90	100
Baker (2)	60	80	90	Leon (3)	60	70	80
Bay (3)	70	80	90	Levy (2)	70	80	90
Bradford (2)	60	80	90	Liberty (3)	60	80	90
Brevard (5)	80	90	100	Madison (2)	60	70	80
Broward (4)	90	100	110	Manatee (1)	80	90	100
Calhoun (3)	60	80	90	Marion (5)	60	80	90
Charlotte (1)	80	90	100	Martin (4)	80	90	100
Citrus (7)	70	80	90	Miami-Dade (6)	90	100	110
Clay (2)	60	80	90	Monroe (6)	90	100	110
Collier (1)	80	90	100	Nassau (2)	70	80	90
Columbia (2)	60	70	80	Okaloosa (3)	70	90	100
DeSoto (1)	70	80	90	Okeechobee (1)	70	80	90
Dixie (2)	70	80	90	Orange (5)	70	80	90
Duval (2)	70	80	90	Osceola (5)	70	80	90
Escambia (3)	70	90	100	Palm Beach (4)	80	100	110
Flagler (5)	70	80	90	Pasco (7)	70	90	100
Franklin (3)	70	90	100	Pinellas (7)	70	90	100
Gadsden (3)	60	70	80	Polk (1)	70	80	90
Gilchrist (2)	60	80	90	Putnam (2)	60	80	90
Glades (1)	70	80	90	St. Johns (2)	70	80	90
Gulf (3)	70	90	100	St. Lucie (4)	80	90	100
Hamilton (2)	60	70	80	Santa Rosa (3)	70	90	100
Hardee (1)	70	80	90	Sarasota (1)	80	90	100
Hendry (1)	70	80	90	Seminole (5)	70	80	90
Hernando (7)	70	90	100	Sumter (5)	60	80	90
Highlands (1)	70	80	90	Suwannee (2)	60	70	80
Hillsborough (7)	70	80	90	Taylor (2)	70	80	90
Holmes (3)	60	70	80	Union (2)	60	80	90
Indian River (4)	80	90	100	Volusia (5)	80	90	100
Jackson (3)	60	70	80	Wakulla (3)	70	80	90
Jefferson (3)	60	70	80	Walton (3)	70	80	90
Lafayette (2)	60	80	90	Washington (3)	60	80	90
Lake (5)	60	80	90				

VOLUME 3 - REVISION HISTORY

Revised cross references to LTS-6 throughout.

3.8.6 ....Revised the requirements of Section; Added paragraph to Commentary; Added new FDOT Figure 3.8.6-1.

5.15.1 ....Clarified Section and Commentary.

5.16 ....Clarified Section and Commentary.

5.17.3.3 ....Deleted reference to monotube structures from second Paragraph.

10.5 ....Clarified camber requirements and revised commentary.

13.6.1.1 ....Revised definition of T_n and related variables; Revised Commentary.

13.6.2.1 ....Clarified reinforcement requirements; Revised Commentary to address use of mass concrete and reinforcement spacing.

C.1 ....Clarified conditions under which a Design Variation/Exception is not required.