CHAPTER 3 – LOADS AND LOAD FACTORS

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3.3—NOTATION

3.3.2—Load and Load Designation

The following shall replace definition of WA in *A3.3.2*.

WA = water load, stream pressure or wave force.

3.4—LOAD FACTORS AND COMBINATIONS

3.4.1—Load Factors and Load Combinations

The following shall supplement *A3.4.1*.

Table 3.4.1-1 shall replace Table *A3.4.1-1*.

3.4.2—Load Factors for Construction Loads

The following shall supplement *A3.4.2*.

A statement shall be included in the contract plan, which clearly defines that the contractor is responsible for safety and stability of structures during all phases of construction and design of forming and bracing systems used to place concrete for bridge components.

A statement shall be included in the contract plan indicating that the contractor is responsible for determining deflection of formwork due to weight of wet concrete, screed and other construction loads.

The EOR shall also consider construction loads during various phases of construction in the design for all applicable load cases.

C3.4.2

The following shall supplement *AC3.4.2*.

The design of formwork and temporary bracing is the contractor's responsibility. The contractor's registered Professional Engineer shall evaluate the ability of all structural elements and formwork to safely support the construction loads. Construction loads shall include but not be limited to forms, bracing, wet concrete, walkway overhangs, workforce, concrete screeding machines and appurtenances. Forming and bracing systems used to place concrete for bridge decks with large overhangs induce large horizontal forces in the fascia girder. These forces can cause lateral buckling and deflection problems in fascia girder resulting in a poor deck profile.

Construction load from temporary material storage on the bridge should be considered if known in the design stage.

Refer to the latest edition of AASHTO *Guide Design Specification for Bridge Temporary Works* for the minimum construction requirements.

Table 3.4.1-1—Load Combinations and Load Factors

										Use One of These at a Time				
Load Combination Limit State	**DC DD DW EH EV ES EL PS CR SH**	**LL IM CE BR PL LS**	**WA**	**WS**	**WL**	**FR**	**TU**	**TG**	**SE**	EQ	IC	CT	CV	**SC**[1]
Strength-I	γ_p	1.75	1.00	-	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-	-
Strength-II	γ_p	1.35	1.00	-	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-	-
Strength-III	γ_p	-	1.00	1.40	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}		-	-	-	-
Strength-IV	γ_p	-	1.00	-	-	1.00	0.50/1.20	-	-	-	-	-	-	-
Strength-V	γ_p	1.35	1.00	0.40	1.00	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-	-
Extreme Event-I	1.00	**0.25**[2]	1.00	-	-	1.00		-	-	1.00	-	-	-	
Extreme Event-II	γ_p	0.50	1.00	-	-	1.00	-	-	-	-	1.00	1.00	1.00	-
Extreme Event-III[1]	$\boldsymbol{\gamma_p}$	**1.75**	**1.00**	**0.30**	-	**1.00**	-	$\boldsymbol{\gamma_{TG}}$	$\boldsymbol{\gamma_{SE}}$	-	-	-	-	**1.00**
Extreme Event-IV[1]	$\boldsymbol{\gamma_p}$	-	**1.00**	**1.40**	-	**1.00**	-	$\boldsymbol{\gamma_{TG}}$	$\boldsymbol{\gamma_{SE}}$	-	-	-	-	**0.70**
Extreme Event-V[1]	$\boldsymbol{\gamma_p}$	-	**1.00**	-	-	**1.00**	-	-	-	-	**1.00**	**1.00**	**1.00**	**0.60**
Extreme Event-VI[1]	$\boldsymbol{\gamma_p}$	-	**1.00**	-	-	**1.00**	-	-	-	**1.00**	-	-	-	**0.25**
Service-I	1.00	1.00	1.00	0.30	1.00	1.00	1.00/1.20	γ_{TG}	γ_{SE}	-	-	-		-
Service-II	1.00	1.30	1.00	-	-	1.00	1.00/1.20	-	-	-	-	-		-
Service-III	1.00	**1.00**[3]	1.00	-	-	1.00	1.00/1.20	γ_{TG}	γ_{SE}	-	-	-		-
Service-IV	1.00	-	1.00	0.70	-	1.00	1.00/1.20	-	1.00	-	-	-		-
Fatigue- I LL, IM & CE only	-	1.50	-	-	-	-	-	-	-	-	-	-	-	-
Fatigue- II LL, IM & CE only	-	0.75	-	-	-	-	-	-	-	-	-	-	-	-

1. SC (Scour) is the total scour depth determined by Bridge Hydraulic Engineer in accordance with *HEC-18*. Scour is not a load, but an extreme event that alters geometry of the foundation, possibly causing structural collapse or amplification of applied load effects. Adopted factors for SC are based on NCHRP Report 489, *Design of Highway Bridges for Extreme Events*, and modified for Louisiana practice.

2. NCHRP Report 489 has shown that the commonly used live load factor of 0.50 in combination with earthquake effects is conservative and a reduced live load factor of 0.25 will provide an adequate safety level. Since probability of a major earthquake occurring in Louisiana is generally very low, it is reasonable to use a live load factor of 0.25.

(*continued on next page)*

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3. The 0.8 factor is based on the performance of bridges designed under LFD criteria which did not include lane load provision in the live load model. In addition, prestress loss calculations have gone through further refinement, resulting in significant reduction of prestressed loss, which results in more streamlined bridges. Another aspect to consider is that current experience from Louisiana and many states indicates a trend towards heavier hauling vehicles which significantly exceeds the HL-93 live load model.

3.5—PERMANENT LOADS

3.5.1—Dead Loads: DC, DW and EV

The following shall supplement *A3.5.1*.

Table 3.5.1-1, which lists the unit weight of common permanent loads, shall replace Table *A3.5.1-1*.

Design for future wearing surface (DW) shall be per Table 3.5.1-1. DW shall not be considered in the computation of girder camber, creep, or geometry. DW shall be distributed equally to all girders in the cross section.

Stay-in-place forms may be used between interior girders and shall be assumed to be simply supported. Stay-in-place forms shall not be used on deck overhangs unless approved by the Bridge Design Engineer Administrator. The unit weight shall be per Table 3.5.1-1 unless a more precise weight is available from the manufacturer.

For bridges without a raised sidewalk, barrier weight and weight of all incidental attachments to barrier, such as fences, sound walls, and minor utilities, shall be distributed equally to all girders in the cross section.

For bridges with a raised sidewalk, the following distributions shall apply:

- If an entire sidewalk is on the overhang, the total weight of barrier and all incidental attachments shall be distributed to the exterior girder only.
- If a sidewalk spans over the exterior girder only, 60 percent of the total weight of barrier and all incidental attachments shall be distributed to the exterior girder, the remaining 40 percent shall be equally distributed to interior girders.
- If a sidewalk spans over two or more girders, the total weight of barrier and all incidental attachments shall be distributed equally to all girders in cross section.

For special cases, such as staged construction or presence of heavy utilities, a more accurate method for distribution of these loads shall be investigated.

Bridges which require installation of sound walls shall be designed to accommodate corresponding dead, live, and wind loads for the required wall height. The minimum weight of wall shall be per Table 3.5.1-1.

Table 3.5.1-1 — Unit Weights of Common Permanent Loads

Aluminum Alloys		0.175 kcf
Asphaltic Concrete, Unit Weight		0.145 kcf
Barrier (32’’ F-Shape)		0.284 klf
Barrier (42’’ F-Shape)		0.521 klf
Barrier (32’’ F-Shape Double Face Median Barrier)		0.437 klf
Barrier (42’’ F-Shape Double Face Median Barrier)		0.585 klf
Barrier (32’’ Vertical Face)		0.333 klf
Barrier (32’’Vertical Face)		0.525 klf
Bituminous Wearing Surfaces		0.140 kcf
Cast Iron		0.450 kcf
Cinder Filling		0.060 kcf
Compacted Sand, Silt, or Clay		0.120 kcf
Concrete Overlay		0.150 kcf
Concrete - Lightweight		0.110 kcf
Concrete – Normal Weight ($f_c' \leq 5.0\ ksi$)		0.145 kcf
Concrete – Normal Weight ($5.0\ ksi < f_c' \leq 15.0\ ksi$)		$0.140 + 0.001\ f_c'$ kcf
Concrete – Reinforced ($f_c' < 7.5\ ksi$)		0.150 kcf
Concrete – Reinforced ($f_c' \geq 7.5\ ksi$)		0.155 kcf
Future Wearing Surface (Between the Curbs)		0.025 ksf
Loose Sand, Silt, or Gravel		0.100 kcf
Rolled Gravel, Macadam, or Ballast		0.140 kcf
Soft Clay		0.100 kcf
Soil (Compacted)		0.125 kcf
Stay-in-Place Metal Forms (Foam Filled)		0.010 ksf
Steel		0.490 kcf
Sound Wall		• Min. 0.100 klf for wall heights up to 10 ft. • Min. 0.200 klf for wall heights greater than 10 ft.
Stone Masonry		0.170 kcf
Transit Rails, Ties, and Fastening per Track		0.200 klf
Wood	Hard	0.060 kcf
	Soft	0.050 kcf
Water	Fresh	0.0624 kcf
	Salt	0.0640 kcf

3.6—LIVE LOADS

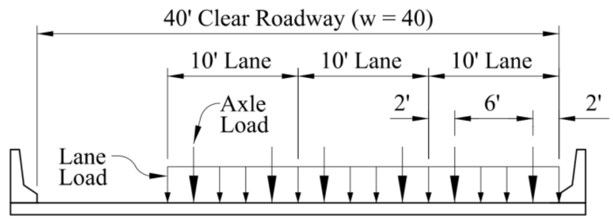
3.6.1—Gravity Loads: *LL* and *PL*

3.6.1.1—Vehicular Live Load

3.6.1.1.1—Number of Design Lanes

The following shall supplement *A3.6.1.1.1*.

Use of 10 feet design lane width is allowed for manual calculations. Live load shall be allowed anywhere on the clear roadway including a raised median with or without a mountable curb.



Number of Design Lanes = 40' / 12 = 3.33 (Use 3)

Width of Design Lane = 10'

<u>Design Lane Example for Manual Calculations</u>

3.6.1.2 Design Vehicular Live Load

3.6.1.2.1 —General

The following shall supplement *A3.6.1.2.1*.

All bridges in Louisiana shall be designed for *Louisiana Design Vehicle Live Load 2011* (LADV-11). Use of LADV-11 shall be indicated on the General Notes plan sheet under "Design Criteria".

LADV-11 is the product of the force effects produced by HL-93, as specified in *A3.6.1.2* and a magnification factor (MF) listed in the Magnification Factor Table below.

MF in the Magnification Factor Table shall be applied to all bridge elements and limit states that are subject to design vehicular live load, but with the following exceptions:

- MF = 1.0, when applying the design vehicular live load to decks, deck systems, and the top slab of box culverts per *A3.6.1.3.3*.
- MF = 1.0, when determining the live load deflection per *A3.6.1.3.2*.

For fatigue load in *A3.6.1.4*, MF in the Magnification Factor Table shall be applied to the design truck. For braking forces in *A3.6.4*, MF in the Magnification Factor Table shall be applied to the design truck or lane load, respectively.

C3.6.1.2.1

The following shall supplement *AC3.6.1.2.1*.

LADV-11 was developed to provide a live load model that is representative of routine permit vehicles in Louisiana, which are not enveloped by the HL-93 load model. Bridges designed using LADV-11 will meet the minimum service and strength requirements for these vehicles and satisfy load rating and evaluation criteria.

Magnification factors were developed through rigorous analysis of the load effects of the aforementioned permit vehicles and HL-93 load model on simple and continuous span bridges with varying span lengths. The value of MF varies and is a function of span length. LADV-11 is essentially a magnified HL-93 load model that is representative of current routine truck traffic in Louisiana.

The study report for the development of LADV-11, "*LADV-11 Development*", is included in *BDEM-Part IV*.

A3.6.1.7 (Loads on Railing), *A3.6.3* (Centrifugal Forces), and *A3.6.5* (Vehicular Collision Forces) will not be impacted by LADV-11 and shall remain unchanged.

LADV-11 Magnification Factor Table

Load Effect	Range of Applicability	Magnification Factor (MF)
M^+, V	$L \leq 240$	1.30
	$240 < L < 600$	1.30-0.00083(L-240)
	$L \geq 600$	1.00
M^-	$L \leq 100$	1.30
	$100 < L < 240$	1.30-0.00214(L-100)
	$L \geq 240$	1.00
R_B	All Span Lengths	1.60
R_F	$L_1 + L_2 \leq 100$	1.30
	$100 < L_1 + L_2 < 240$	$1.30-0.00214(L_1+L_2-100)$
	$L_1 + L_2 \geq 240$	1.00
R_S	$L_1 + L_2 \leq 100$	1.55
	$100 < L_1 + L_2 < 600$	$1.55-0.00110(L_1+L_2-100)$
	$L_1 + L_2 \geq 600$	1.00

L = Span Length taken as center of bearing to center of bearing length, feet (use the shortest span length for unequal continuous spans)

$L_1 + L_2$ = Sum of Span 1 Length and Span 2 Length on either side of the support, feet (for end bents use the approach slab length as L_1 and the span length as L_2)

M^+ = Positive Moment (use for design of superstructure elements only)

M^- = Negative Moment (use for design of superstructure elements only)

V = Shear (use for design of superstructure elements only)

R_B = Bearing Reaction (use for design of bearings only)

R_F = Factored Support Reaction (use for design of all substructure elements and determination of factored pile/shaft loads)

R_S = Service Support Reaction (use for determination of service pile/shaft loads only)

* Equations are linear interpolations between the upper and lower values of the MFs.

3.6.1.3—Application of Design Vehicular Live Loads

3.6.1.3.1—General

The following shall supplement *A3.6.1.3.1*.

For slab span bents, the live load shall be placed along span to cause a maximum reaction to the bent. The wheel loads directly over the bent shall be treated as concentrated loads. Contributing reactions from wheel loads on span and lane load shall be treated as uniform load distributed over the design lane width of 10 ft.

C3.6.1.3.1

The following shall supplement *AC3.6.1.3.1*.

An interior pier or interior support is defined as a substructure supporting a continuous segment of superstructure.

3.6.1.3.4 —Deck Overhang Load

The following shall replace first paragraph of *A 3.6.1.3.4.*

Structural stiffness and strength contribution of continuous barriers shall not be considered in the design of any structural bridge component.

3.6.1.4 —Fatigue Load

3.6.1.4.2 —Frequency

C3.6.1.4.2

The following shall supplement *AC3.6.1.4.2*.

A typical traffic data sheet prepared by LADOTD Traffic Engineer consists of the following information:

Current year ADT = Average daily traffic volume for both directions of travel
20 year design life ADT = Average daily traffic volume for both directions of travel for 20 year design life
R = Annual Growth Rate (%)
D = Directional distribution factor (%)
T = Percentage of truck traffic (%)

Frequency of the fatigue load $ADTT_{SL}$ that averages over the bridge design life of 75 years should be determined utilizing information in traffic data sheet as follows:

Example 1:

Assume current year ADT = 5000
R = 2%
D = 55%
T = 12%
Number of lanes per direction available to trucks = 2
(If there is a designated truck lane, this number should be taken as 1.)

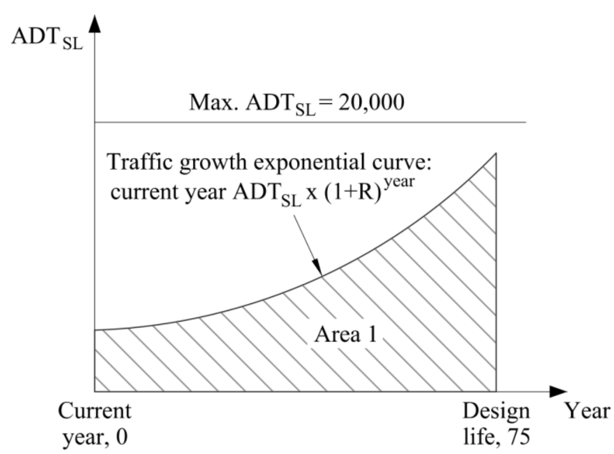
Step 1: Determine current year ADT_{SL} per direction per lane.

Current year ADT_{SL} = current year ADT × D × p (per *Table A3.6.1.4.2-1*) = 5000 × 55% × 0.85 = 2,338

Step 2: Determine 75 year design life ADT per direction per lane using the annual growth rate, this number should not exceed 20,000 per *AC3.6.1.4.2*.

75 year design life ADT_{SL} = current year ADT_{SL} × $(1+R)^{75}$ = 2,338 × $(1+2\%)^{75}$ = 10,324 < 20,000

Step 3: Determine average ADT_{SL} over 75 years of design life (equal to area 1 under exponential curve of traffic growth from current year to 75 year divided by a design life span of 75 years).



Area 1 under exponential curve of traffic growth from current year to 75 year =
(current ADT_{SL})×((1+R)^{75}-1)/(ln(1+R)) = 2,338×((1+2%)^{75} -1)/(ln(1+2%)) = 403,291
Average ADT_{SL} = 403,291/75 = 5,377

Step 4: Determine average $ADTT_{SL}$ over 75 years of design life.

Average $ADTT_{SL}$ = Average ADT_{SL} × T = 5,377 × 12% = 645

Example 2:

Assume current year ADT = 10,000
R = 2%
D = 55%
T = 12%
Number of lanes per direction available to trucks = 1
(Assume there is a designated truck lane.)

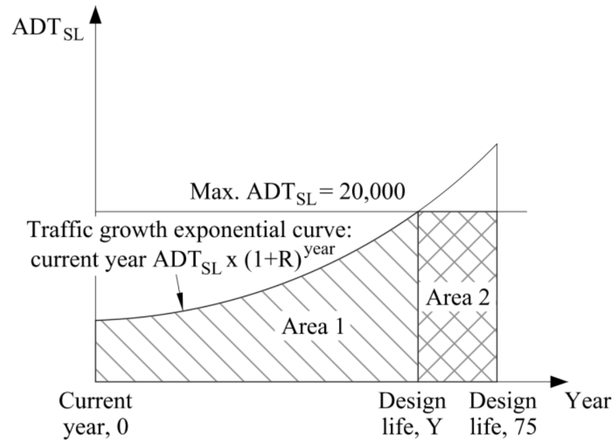
Step 1: Determine current year ADT_{SL} per direction per lane.

Current year ADT_{SL} = current year ADT × D × p (per *Table A3.6.1.4.2-1*) = 10,000 × 55% × 1 = 5,500

Step 2: Determine 75 year design life ADT per direction per lane using the annual growth rate; this number should not exceed 20,000 per AC3.6.1.4.2.

75 year design life ADT_{SL} = current year ADT_{SL} × (1+R)^{75} = 5,500 × (1+2%)^{75} = 24,287 > 20,000
Max. design life ADT_{SL} = 20,000

Step 3: Determine the design year, Y, that reaches maximum design life ADT_{SL} = 20,000.



Solve Y from this equation:
Max. design life ADT_{SL} = current $ADT_{SL} \times (1+R)^Y$
$20{,}000 = 5500\ (1+2\%)^Y$
Y = 65 years

Step 4: Determine average ADT_{SL} over 75 years of design life (equal to the sum of area 1 under exponential curve of traffic growth from current year to year 65 and area 2 under maximum design life ADT_{SL} from year 65 to year 75 divided by a design life span of 75 years).

Area 1 under exponential curve of traffic growth from current year to year 65=
$(\text{current } ADT_{SL})\times((1+R)^{65}-1)/(\ln(1+R)) = 5{,}500\times((1+2\%)^{65}-1)/(\ln(1+2\%)) = 728{,}382$

Area 2 under maximum design life ADT_{SL} from year 65 to year 75 =
$20{,}000 \times (75-65) = 200{,}000$

Average $ADT_{SL} = (728{,}382 + 200{,}000)/75 = 12{,}378$

Step 5: Determine average $ADTT_{SL}$ over 75 years of design life.

Average $ADTT_{SL}$ = Average $ADT_{SL} \times T = 12{,}378 \times 12\% = 1{,}485$

3.6.1.4.3—Load Distribution for Fatigue

3.6.1.4.3a—Refined Methods

The following shall supplement *A3.6.1.4.3a.*

The live load distribution factors shall be calculated based on the refined analysis and clearly shown on design plans.

3.6.2—Dynamic Load Allowance: *IM*

3.6.2.1—General

The following shall supplement the last paragraph of *A3.6.2.1*.

For piles or drilled shafts that are not fully embedded in the ground, IM shall be included in the structural capacity check but not in the load calculations used for length determination.

3.6.4—Braking Force: BR

The following shall supplement *A3.6.4.*

For two-directional bridges that are not likely to become one-directional in the future, the number of lanes used to calculate the braking force shall be determined by dividing the total number of design lanes by two and rounding to the nearest integer.

For bridges that are designed as one-directional, or that are likely to become one-directional in the future, the number of lanes used to calculate the braking force shall be equal to the total number of design lanes.

Dynamic load allowance shall not be applied to the braking force.

For superstructures supported by combination of a fixed bearing and expansion bearings, braking force shall be transferred to substructures by the fixed bearing only when practical. If the substructure at fixed bearing is not able to take the total braking force, it is allowed to distribute a portion of the braking force to the adjacent expansion bearings and its substructures. The displacement of the expansion bearings caused by the braking force alone shall not exceed 20% of the design thermal movement of the expansion bearings. For superstructures supported by expansion bearings only, braking force shall be equally distributed to all expansion bearings. The additional displacement requirement due to the braking force shall be included in the bearing design. The braking force and the effect of the moment component created by the braking force applied at 6 feet above the roadway surface shall be considered in the design of bearings and substructures.

3.6.5.1 - Protection of Structures

The following shall supplement *A3.6.5.1*.

The provisions in *A3.6.5.1* shall be followed unless otherwise approved by the Bridge Design Engineer Administrator for specific project sites.

3.7—WATER LOADS: WA

3.7.3—Stream Pressure

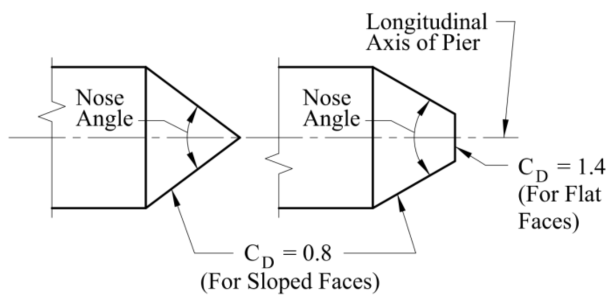
3.7.3.1—Longitudinal

The following shall supplement *A3.7.3.1*.

The debris raft as shown in Figure *AC3.7.3.1-1* shall be applied to all streams with known exhibition of or potential for debris build-up.

C3.7.3.1

The following shall supplement *AC3.7.3.1*.



<u>Plan View of Wedged-Nosed Piers</u>

3.7.3.2—Lateral

The following shall supplement *A3.7.3.2.*

To allow for a change in the direction of flow over the life of the structure, add an additional 5 degrees to the angle between direction of flow and longitudinal axis of the pier when determining the lateral drag coefficient per Table *A3.7.3.2-1*.

3.7.4—Wave Load

The following shall supplement *A3.7.4.*

For bridges identified as vulnerable to coastal storms, storm surge and wave forces shall be developed based on the latest AASHTO *Guide Specification for Bridges Vulnerable to Coastal Storms*.

3.8—WIND LOAD: *WL* AND *WS*

3.8.3—Aeroelastic Instability

3.8.3.1—General

The following shall supplement *A3.8.3.1*.

For cable-stayed bridges, determination of aeroelastic instability shall be based on wind tunnel tests performed during design phase.

3.9—ICE LOADS: *IC*

The following shall supplement *A3.9.*

This section is not applicable to design of bridges in Louisiana.

3.10—EARTHQUAKE EFFECTS: *EQ*

The following shall supplement *A3.10.*

The following preliminary seismic design information has been developed based on the 1984 Geologic Map of Louisiana. Bridge designer shall work with Geotechnical Engineer to finalize the design information.

- Louisiana Seismic Site Class Map per *A3.10.3.1*.
- Louisiana Seismic Zone Map per *A3.10.6.*
- Louisiana Seismic Design Information Table for each parish. The information includes peak ground acceleration coefficient (PGA per *A3.10.2.1*), short- and long-period spectral acceleration coefficients (S_S & S_1 per *A3.10.2.1*), site class, site factors (F_{pga}, F_a & F_v per *A3.10.3.2*) and elastic seismic response coefficients (A_S, S_{DS} &S_{D1} per *A3.10.4.2*).

C3.10

The following shall supplement *AC3.10*.

The AASHTO site classes A though F are defined in *Table 3.10.3.1-1*. The classifications are based on weighted average soil conditions for the upper 100 feet of soil profile with the exception of site class F with very thick soft/medium stiff clays.

The 1984 Geological Map of Louisiana shows the geological ages of the surficial soils. Since ages of soils can be strongly correlated to the soil strengths, it is appropriate to use the geological map as the basis for site classification.

Site classes A and B are for medium to hard rocks. The occasional outcrop of rocks in Louisiana is insignificant to be used for site classification using Parish as mapping unit. As such, the site class map does not contain classes A and B sites.

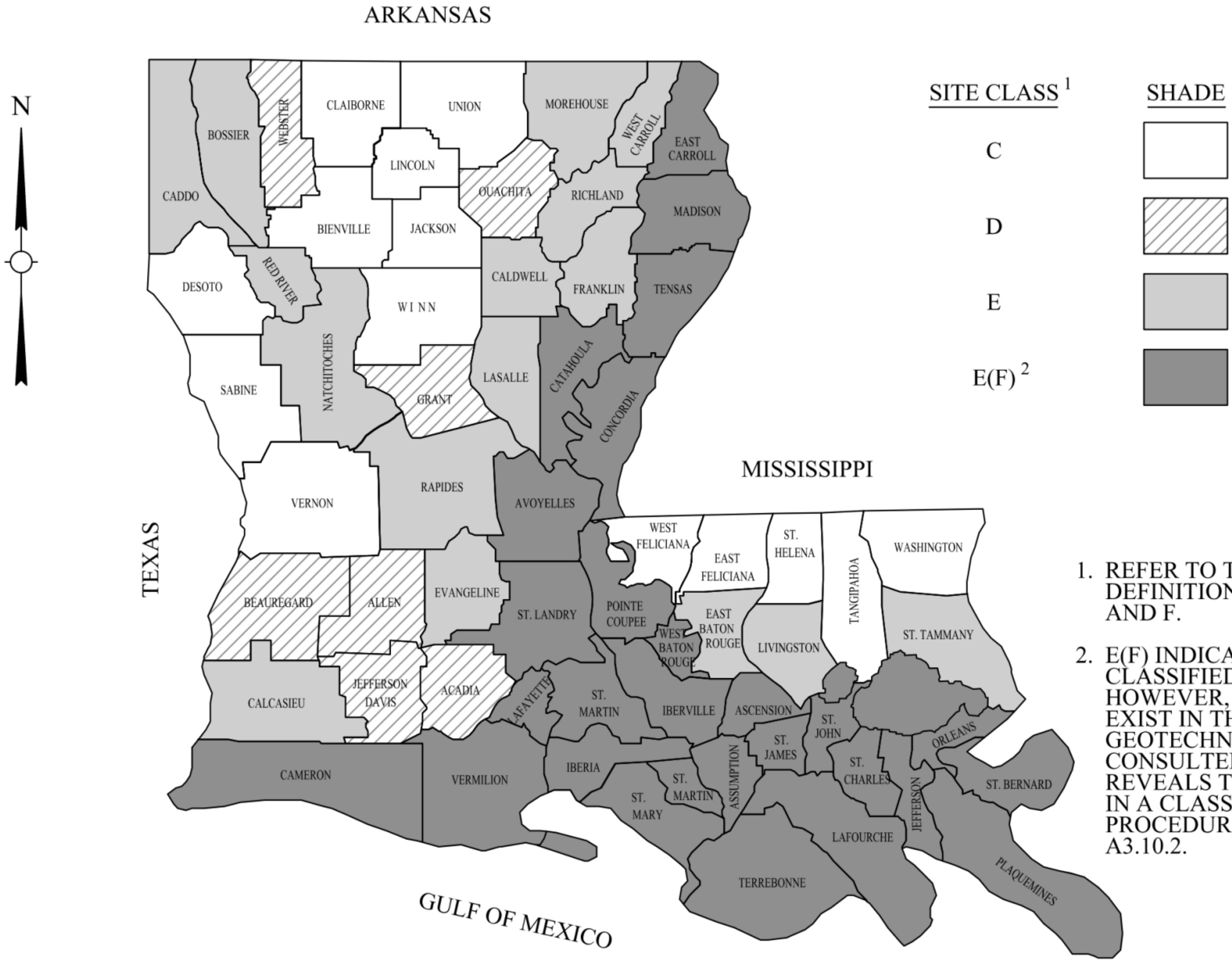
It is assumed that the soils that are Miocene and older have strengths greater than 2 ksf and these parishes are grouped as Site Class C.

The soils at Webster, Ouachita and Grant parishes are quite strong and can be grouped in the Site Class C. However, the tributaries of Red River, Ouachita River, and Little River cut through significant part of the parishes. The waterways brought soils that are unconsolidated and are much weaker. Since most bridges are constructed for waterway crossing, these parishes are grouped as Site Class D.

The soils deposited in Pliocene age are also quite strong. For parishes where no major waterway exists, they are classified as Site D. These parishes include Acadia, Allen, Beauregard, East Feliciana, Jefferson, St. Helena, Tangipahoa, Washington, and West Feliciana.

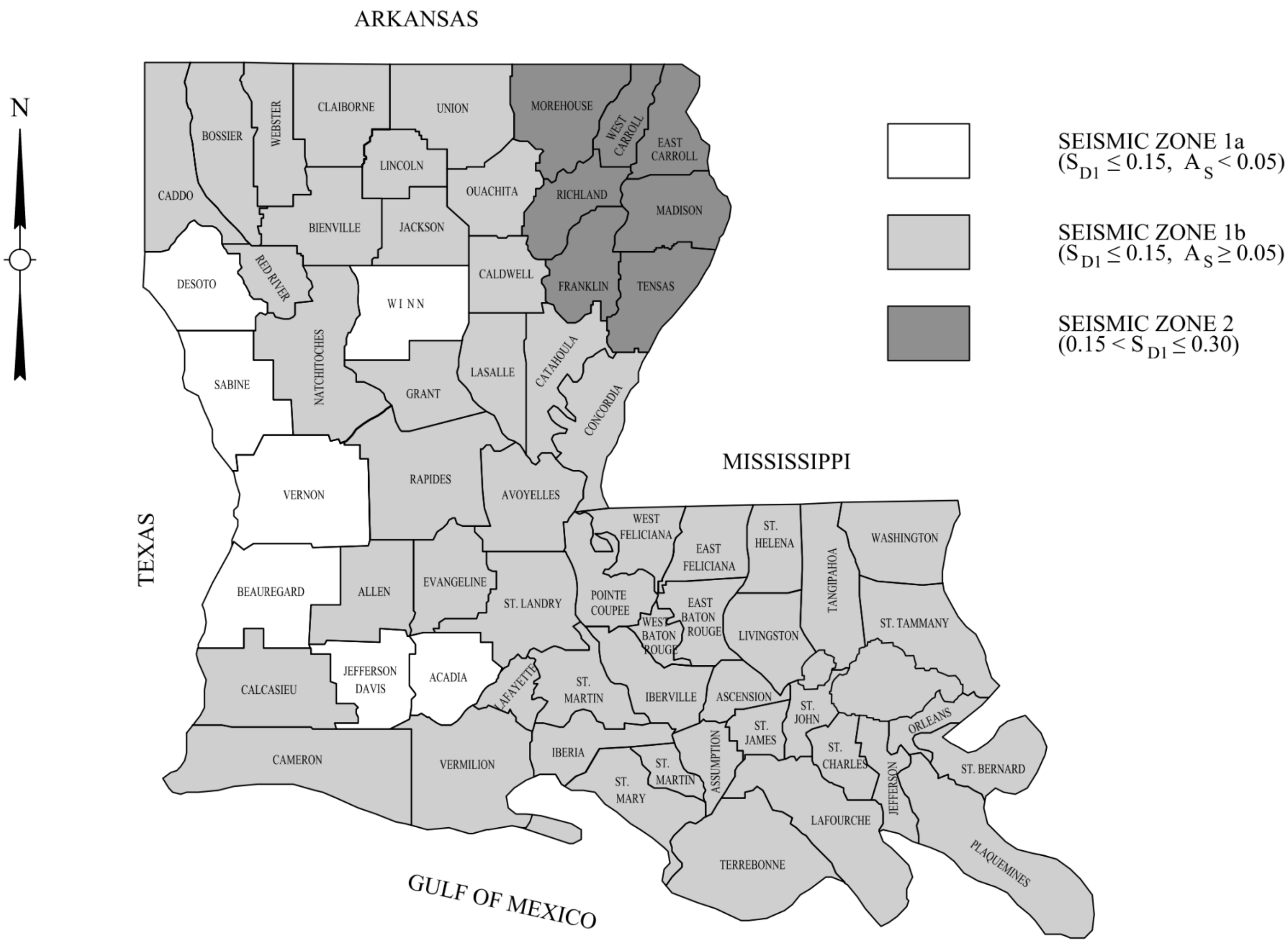
The dominant geologies of the remaining parishes are Pleistocene or younger. These parishes fall under the site classification of E. It should be noted that the parishes with majority of the areas that are part of the Mississippi River, the Atchafalaya River, and coastal zones are rich in organics and are also grouped in Site E. However, the specific bridge sites may contain significant organic material and may be classified as Site F. The geotechnical engineer of the record should verify the site classifications within these parishes.

It is very important to note that the above site classifications are based on the generalized geological map. It is likely that the site soils have been modified by local stream or human activities. The project geotechnical engineer should always verify the site classifications based on actual geotechnical data gathered.



1. REFER TO TABLE A3.10.3.1-1 FOR THE DEFINITION OF SITE CLASSES C, D, E AND F.
2. E(F) INDICATES THESE PARISHES ARE CLASSIFIED AS SITE CLASS E IN GENERAL. HOWEVER, CLASS F CONDITIONS MAY EXIST IN THESE PARISHES. DOTD GEOTECHNICAL SECTION SHALL BE CONSULTED IF THE BORING INFORMATION REVEALS THE BRIDGES MAY BE LOCATED IN A CLASS F CONDITION. A SITE SPECIFIC PROCEDURE MAY BE REQUIRED PER A3.10.2.

<u>Louisiana Seismic Site Class Map</u>



Louisiana Seismic Zone Map

Louisiana Seismic Design Information Table

Parish	PGA	S_s	S_1	Site Class	F_{pga}	F_a	F_v	$A_s=F_{pga}$ PGA	$S_{DS}=F_a S_s$	$S_{D1}=F_v S_1$	Zone
Acadia	0.027	0.06	0.028	D	1.6	1.6	2.4	0.043	0.096	0.070	1a
Allen	0.031	0.07	0.032	D	1.6	1.6	2.4	0.050	0.112	0.080	1b
Ascension	0.029	0.065	0.03	E (F)	2.5	2.5	3.5	0.073	0.163	0.110	1b
Assumption	0.027	0.058	0.027	E (F)	2.5	2.5	3.5	0.068	0.145	0.090	1b
Beauregard	0.03	0.067	0.03	D	1.6	1.6	2.4	0.048	0.107	0.070	1a
Bienville	0.043	0.1	0.044	C	1.2	1.2	1.7	0.052	0.120	0.070	1b
Bossier	0.044	0.101	0.044	E	2.5	2.5	3.5	0.110	0.253	0.150	1b
Caddo	0.042	0.096	0.042	E	2.5	2.5	3.5	0.105	0.240	0.150	1b
Calcasieu	0.027	0.06	0.027	E	2.5	2.5	3.5	0.068	0.150	0.090	1b
Caldwell	0.042	0.098	0.044	E	2.5	2.5	3.5	0.105	0.245	0.150	1b
Cameron	0.025	0.056	0.026	E (F)	2.5	2.5	3.5	0.063	0.140	0.090	1b
Catahoula	0.039	0.091	0.042	E (F)	2.5	2.5	3.5	0.098	0.228	0.150	1b
Claiborne	0.049	0.114	0.048	C	1.2	1.2	1.7	0.059	0.137	0.080	1b
Concordia	0.037	0.086	0.04	E (F)	2.5	2.5	3.5	0.093	0.215	0.140	1b
Desoto	0.037	0.084	0.037	C	1.2	1.2	1.7	0.044	0.101	0.060	1a
East Baton Rouge	0.031	0.069	0.032	E	2.5	2.5	3.5	0.078	0.173	0.110	1b
East Carroll	0.058	0.138	0.056	E (F)	2.5	2.5	3.5	0.145	0.345	0.200	2
East Feliciana	0.041	0.094	0.041	D	1.6	1.6	2.4	0.066	0.150	0.100	1b
Evangeline	0.029	0.065	0.031	E	2.5	2.5	3.5	0.073	0.163	0.110	1b
Franklin	0.048	0.115	0.049	E	2.5	2.5	3.5	0.120	0.288	0.170	2
Grant	0.037	0.085	0.039	D	1.6	1.6	2.4	0.059	0.136	0.090	1b
Iberia	0.026	0.058	0.027	E (F)	2.5	2.5	3.5	0.065	0.145	0.090	1b
Iberville	0.029	0.065	0.03	E (F)	2.5	2.5	3.5	0.073	0.163	0.110	1b
Jackson	0.044	0.103	0.045	C	1.2	1.2	1.7	0.053	0.124	0.080	1b
Jefferson	0.027	0.059	0.027	E (F)	2.5	2.5	3.5	0.068	0.148	0.090	1b
Jefferson Davis	0.028	0.062	0.029	D	1.6	1.6	2.4	0.045	0.099	0.070	1a

Louisiana Seismic Design Information Table (Continued)

Parish	PGA	S_s	S_1	Site Class	F_{pga}	F_a	F_v	A_s=F_{pga} PGA	S_{DS}=F_a S_s	S_{D1}=F_v S_1	Zone
Lafayette	0.028	0.061	0.029	E (F)	2.5	2.5	3.5	0.070	0.153	0.100	1b
Lafourche	0.026	0.057	0.026	E (F)	2.5	2.5	3.5	0.065	0.143	0.090	1b
LaSalle	0.039	0.091	0.041	E	2.5	2.5	3.5	0.098	0.228	0.140	1b
Lincoln	0.048	0.113	0.048	C	1.2	1.2	1.7	0.058	0.136	0.080	1b
Livingston	0.031	0.068	0.031	E	2.5	2.5	3.5	0.078	0.170	0.110	1b
Madison	0.049	0.116	0.049	E (F)	2.5	2.5	3.5	0.123	0.290	0.170	2
Morehouse	0.061	0.144	0.057	E	2.5	2.5	3.5	0.153	0.360	0.200	2
Natchitoches	0.038	0.088	0.04	E	2.5	2.5	3.5	0.095	0.220	0.140	1b
Orleans	0.028	0.06	0.028	E (F)	2.5	2.5	3.5	0.070	0.150	0.100	1b
Ouachita	0.048	0.114	0.049	D	1.6	1.6	2.4	0.077	0.182	0.120	1b
Plaquemines	0.026	0.057	0.027	E (F)	2.5	2.5	3.5	0.065	0.143	0.090	1b
Pointe Coupee	0.031	0.07	0.033	E (F)	2.5	2.5	3.5	0.078	0.175	0.120	1b
Rapides	0.033	0.077	0.036	E	2.5	2.5	3.5	0.083	0.193	0.130	1b
Red River	0.039	0.091	0.04	E	2.5	2.5	3.5	0.098	0.228	0.140	1b
Richland	0.048	0.115	0.049	E	2.5	2.5	3.5	0.120	0.288	0.170	2
Sabine	0.036	0.081	0.035	C	1.2	1.2	1.7	0.043	0.097	0.060	1a
St. Bernard	0.027	0.059	0.026	E (F)	2.5	2.5	3.5	0.068	0.148	0.090	1b
St. Charles	0.027	0.06	0.028	E (F)	2.5	2.5	3.5	0.068	0.150	0.100	1b
St. Helena	0.032	0.073	0.034	D	1.6	1.6	2.4	0.051	0.117	0.080	1b
St. James	0.028	0.062	0.028	E (F)	2.5	2.5	3.5	0.070	0.155	0.100	1b
St. John	0.028	0.062	0.028	E (F)	2.5	2.5	3.5	0.070	0.155	0.100	1b
St. Landry	0.028	0.064	0.03	E (F)	2.5	2.5	3.5	0.070	0.160	0.110	1b
St. Martin	0.028	0.061	0.029	E (F)	2.5	2.5	3.5	0.070	0.153	0.100	1b
St. Mary	0.026	0.057	0.027	E (F)	2.5	2.5	3.5	0.065	0.143	0.090	1b
St. Tammany	0.031	0.069	0.032	E	2.5	2.5	3.5	0.078	0.173	0.110	1b
Tangipahoa	0.032	0.073	0.034	D	1.6	1.6	2.4	0.051	0.117	0.080	1b
Tensas	0.043	0.1	0.045	E (F)	2.5	2.5	3.5	0.108	0.250	0.160	2

Louisiana Seismic Design Information Table (Continued)

Parish	PGA	S_s	S_1	Site Class	F_{pga}	F_a	F_v	$A_s=F_{pga}$ PGA	$S_{DS}=F_a S_s$	$S_{D1}=F_v S_1$	Zone
Terrebonne	0.024	0.053	0.025	E (F)	2.5	2.5	3.5	0.060	0.133	0.090	1b
Union	0.055	0.13	0.053	C	1.2	1.2	1.7	0.066	0.156	0.090	1b
Vermilion	0.025	0.056	0.026	E (F)	2.5	2.5	3.5	0.063	0.140	0.090	1b
Vernon	0.033	0.073	0.033	C	1.2	1.2	1.7	0.040	0.088	0.060	1a
Washington	0.033	0.074	0.035	D	1.6	1.6	2.4	0.053	0.118	0.080	1b
Webster	0.047	0.109	0.046	D	1.6	1.6	2.4	0.075	0.174	0.110	1b
West Baton Rouge	0.03	0.067	0.031	E (F)	2.5	2.5	3.5	0.075	0.168	0.110	1b
West Carroll	0.059	0.14	0.056	E	2.5	2.5	3.5	0.148	0.350	0.200	2
West Feliciana	0.031	0.071	0.033	D	1.6	1.6	2.4	0.050	0.114	0.080	1b
Winn	0.04	0.093	0.042	C	1.2	1.2	1.7	0.048	0.112	0.070	1a

3.10.5—Operational Classification

The following shall supplement *A3.10.5.*

All bridges shall be classified as "Other Bridges," except those on the National Highway System where no detour exists within 5 miles shall be classified as "Essential Bridges." Critical Bridges may be classified at the direction of the Bridge Design Engineer Administrator for specific projects.

3.10.8—Combination of Seismic Force Effects

The following shall replace last paragraph of *A3.10.8.*

Plastic hinging of the columns as specified in *A3.10.9.4.3* is not allowed as a basis for seismic design.

3.12—FORCE EFFECTS DUE TO SUPERIMPOSED DEFORMATIONS: *TU, TG, SH, CR, SE, PS*

3.12.2—Uniform Temperature

The following shall replace *A3.12.2.*

For all bridge types, design value for thermal movement associated with uniform temperature change shall be calculated according to *D.3.12.2.1.*

3.12.2.1—Temperature Range for Procedure A

The following shall replace *A3.12.2.1.*

Base construction temperature assumed in design shall be taken as 68°F and the ranges of temperature shall be as specified below:

C3.12.2.1

The following shall supplement *AC3.12.2.1.*

Defined ranges of temperature variation are based on research of Louisiana weather data from 2000 to 2012. The temperature range study report is included in *BDEM, Part IV*.

Design Temperature Range Table

Material	Temperature Range	Rise	Fall	Minimum Temperature	Maximum Temperature
Concrete Girder Bridges	85°F	35°F	50°F	18°F	103°F
Steel Girder Bridges	120°F	52°F	68°F	0°F	120°F

The "rise" and "fall" temperature changes (or the difference from maximum and minimum temperature to base construction temperature) shall be used for thermal deformation effects, however, for design of integral abutments, temperature range (or the difference of the maximum and minimum temperature) shall be used to calculate thermal deformation effects, as the temperature when abutments are constructed may be at the extreme low or the extreme high.

Minimum and maximum temperatures shall be taken as $T_{MinDesign}$ and $T_{MaxDesign}$ in *Eq. A3.12.2.3-1*.

3.12.2.3—Design Thermal Movement

The following shall supplement *A3.12.2.3*.

Coefficients of thermal expansion for concrete and steel are defined in *A5.4* and *A6.4*.

Thermal movements and forces due to restraint from movement shall be considered in all directions.

Force effects resulting from thermal movement at bearings shall be considered in the substructure design, including piles.

Horizontal forces and moments induced in the bridge by restraint of movement at bearings shall be determined in accordance with *A14.6.3*.

C3.12.2.3

The following shall supplement *AC3.12.2.3*.

Note that a given temperature change causes thermal movement in all directions. Because thermal movement is a function of expansion length, a short, wide bridge (for example, a wide and continuous slab span bridge) may experience greater transverse stress than longitudinal stress.

3.12.3—Temperature Gradient

C3.12.4

Past experience in Louisiana has shown that neglecting temperature gradient in the design of continuous concrete and steel girder bridges has not lead to structural distress. In the case of continuous segmental girder bridges the stresses due to the temperature gradient could be significant and should be considered in the design.

3.12.5—Creep

The following shall supplement *A3.12.5*.

The following values shall be used in lieu of AASHTO provisions for estimating movements due to shrinkage and creep:

- 1 inch per 325 feet for concrete prestressed girder bridges (For continuous deck concrete prestressed girder spans, a reduction factor of 0.5 shall be applied.)

C3.12.5

The following shall supplement *C3.12.5*.

This design criterion is based on past LADOTD experience.

- ½ inch per 325 feet for steel girder bridges

For other structures not listed, AASHTO provisions shall be followed.

3.14—VESSEL COLLISION

3.14.1—General

The following shall supplement *A3.14.1*.

Refer to *D2.3.2.2.5* for additional information on design policies for vessel collisions.

The definition of navigational waterways can be found in the bridge permits section of the U.S. Coast Guard website.

The following two vessel collision events shall be evaluated for Extreme II and V Limit States in accordance with Table 3.4.1-1.

- A drifting empty barge breaking loose from its moorings and striking the bridge. Water surface velocity and corresponding water level shall be associated with the maximum historical flood event, but no less than the 100-year flood event. This information shall be determined by the Hydraulic Engineer.
- A ship or barge tow striking the bridge while transiting the navigation channel under typical waterway conditions. Water surface velocity and corresponding water level shall be taken as the maximum values anticipated in which navigation is permitted. This water level is sometimes referred to as the 2% flow line. In absence of this data, values can be determined based on the 50-year flood event. This information shall be determined by the Hydraulic Engineer.

3.14.2—Owner's Responsibility

The following shall supplement *A3.14.2*.

Bridge operational classification shall be established by the Bridge Design Engineer Administrator based on current data.

Vessel collision risk assessment studies for major navigable waterways in Louisiana have been previously performed and information may be available. Contact LADOTD for navigable waterway information.

3.14.3—Operational Classification

The following shall supplement *A3.14.3.*

All bridges shall be designed as "regular" unless defined by the Bridge Design Engineer Administrator as "critical".

3.14.5—Annual Frequency of Collapse

3.14.5.3—Geometric Probability

The following shall supplement *A3.14.5.3*.

Bridge components located beyond 3 times LOA from centerline of the vessel transit path or beyond edge of the waterway do not need to be included in the analysis other than the minimum impact requirement of *A3.14.1*. When waterway width is less than 6 times LOA, the standard deviation of normal distribution to model the sailing path of an aberrant vessel can be taken as one sixth of waterway width.

3.14.6—Design Collision Velocity

The following shall supplement *A3.14.6.*

V_{MIN} shall be taken as the water surface velocity as specified in *D3.14.1* for two vessel collision events.

3.14.13—Damage at Extreme Limit State

The following shall replace *A3.14.13.*

All bridges shall be designed to withstand the impact loads in an elastic manner. Deviation from this policy shall be approved by the Bridge Design Engineer Administrator on a case-by-case basis. Refer to *D2.3.2.2.5* for additional information on design policy for vessel collisions.

3.14.14—Application of Impact Force

3.14.14.1 - Substructure Design

The following shall supplement *A3.14.14.1*.

All columns shall be protected or safeguarded from ship and barge bow collision forces by providing strut walls, raising footings or other means. For consistency, all columns shall have protection to the same elevation. Refer to *D.3.14.1* for additional information on the water level to be used for two vessel collision events. Elevation for column protection shall be determined based on the most critical water level.

3.14.16—Security Considerations

The following shall supplement *A3.14.16.*

Intentional vessel collision event shall not be considered in design unless requested by the Bridge Design Engineer Administrator for specific projects.

3.16—REFERENCES

Guide Design Specifications for Bridge Temporary Works, Latest Edition, American Association of State Highway and Transportation Officials, Washington, DC.

Guide Specification for Bridges Vulnerable to Coastal Storms, Latest Edition, American Association of State Highway and Transportation Officials, Washington, DC.

Design of Highway Bridges for Extreme Events, NCHRP Report 489, Latest Edition, Transportation Research Board, Washington, DC.