BRIDGE DESIGN GUIDELINES

Government of Guam
Department of Public Works
and
US Department of Transportation
Federal Highway Administration

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1.0 INTRODUCTION

This document provides the requirements, standards and guidance to those involved in preparing the designs for bridges and other highway related structures for the Department of Public Works (DPW). The intent of this document is to provide framework to experienced engineers for their use in producing a complete set of bid documents for bridges and highway related structures. This document is not a textbook, nor a substitute for engineering knowledge, experience or judgment. No attempt is made to detail AASHTO Code Requirements or basic engineering techniques; for these, AASHTO LRFD Manual and standard textbooks should be used.

The requirements, standards and processes indicated in this document if implemented effectively should assist in the production of quality construction documents which are prerequisites for building any structure efficiently and economically.

Any comments or suggestions you may have to better these guidelines should be addressed to the Director of Public Works, 542 North Marine Corps Drive, Tamuning GU 96913.

2.0 REQUIREMENTS AND STANDARDS

2.1 Design

2.1.1 Specifications

The following specifications and manuals, including current revisions, apply to all bridge projects as appropriate:

- *AASHTO LRFD Bridge Design Specifications*, American Association of State Highway and Transportation Officials (AASHTO), current edition, Customary U.S. Units, with current interim revisions[1].
- *AASHTO Manual for Bridge Evaluation,* American Association of State Highway and Transportation Officials, current edition
- *Guide Specifications for Bridges Vulnerable to Coastal Storms*, American Association of State Highway and Transportation Officials, 2008.
- *Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals*, American Association of State Highway and Transportation Officials (AASHTO), current edition.
- Bridge Welding Code, AASHTO/AWS D1.5, current edition.
- *NCHRP Report 350, "Recommended Procedures for the Safety Performance Evaluation of Highway Features"*, National Cooperative Highway Research Program.
- *AASHTO Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges*, American Association of State Highway and Transportation Officials (AASHTO), current edition with interim revisions.
- *Guide Specifications for LRFD Seismic Bridge Design,* published by the American Association of State Highway and Transportation Officials (AASHTO), current edition.
- *Seismic Retrofitting Manual for Highway Structures,* published by the Federal Highway Administration, 2005

2.1.2 Guam Specific Requirements

The following Guam specific requirements supplement and in some cases modify their respective portion of the Design Specification.

[1] Interim Specifications are published annually by AASHTO and have the same status as the LRFD Bridge Design Specifications however; they will not be applied to projects retroactively.

2.1.2.1 Live Loading

For new vehicular structures, use the controlling vehicle of the following live loads:

<u>All Routes</u>
Service and Strength I Limit States:
Design Live Load:

- HL-93 per LRFD 3.6.1.3
- Future wearing surface of 30 psf

<u>Additionally, for Routes – 1, 3, 8, 11, 16, 27</u>
Strength II Limit State:
Military Vehicles (See Appendix A)

- MTVR Tractor, Semitrailer Refueler
- MK48/16/870/D7LVSD7
- MTVR, Engineer Equipment Trailer
- MTVR Wrecker, MTVR Wrecker
- LVS MK48/18
- LVS Wrecker/LVS Wrecker

2.1.2.2 Bridge Railing and Approach Guide Railing

All bridge structures require the use of crash tested railing meeting the loading requirements of TL-3 as defined by NCHRP report 350. For bridges without sidewalks, concrete deflector parapets should be used. For bridges with sidewalks, the use of CALTRANS Standard Drawings for Concrete Barrier Type 26 (B11-54) and Tubular Hand Railing (B11-51) is encouraged.

Connect all bridge railing to approach guide rail.

2.1.2.3 Corrosion Protection

For the design of every structural element, corrosion is a significant design consideration, requiring a level of effort to prevent reinforcing steel corrosion.

Reinforcing steel- Provide the following minimum protection system for cast-in-place and pre-cast concrete:

- Decks: Epoxy coat all bars.
- Cross beams, columns, footings and foundations: Epoxy coat all bars.
- Conventionally reinforced concrete beams, girders, slabs, boxes and precast piles: Epoxy coat all bars.
- Prestressed or post-tensioned reinforced concrete beams, girders, slabs, boxes and pretensioned concrete piles: Provide uncoated ("Black") bars for all reinforcing steel. Provide uncoated prestress and post-tensioning strand. Require use of corrosion inhibitor admixture for all precast concrete.

Provide a minimum of 2 inches of cover over reinforcing.

Additional protection measures including concrete sealers, cathodic protection or others should be considered on a project-by-project basis.

2.1.2.4 Seismic

Design all bridges for full seismic loading according to the *AASHTO LRFD Bridge Design Specifications*. Additional information, clarification and explanations for technical basis for seismic design are available in Guide Specifications for LRFD Seismic Bridge Design.

Use mapped spectral response acceleration parameter at 1-sec period equal to 0.60g and at 0.20 sec period equal to 1.50g. Pending the release of the recommended peak ground acceleration for Guam by AASHTO, U.S. Geological Survey's estimation of 7% in 75 years probability of exceedance of 0.34g shall be used for Guam.

Where soil properties are not known in sufficient detail to determine site class, Site class D shall be used unless geotechnical data determines site class E or F soils are present at the site. Design of deep foundations shall consider additional lateral and vertical (downdrag) forces that result from liquefaction if potential for liquefaction exists at the bridge site.

Select site class and site factors F_{pga}, F_a and F_v from tables on the following page.

<table>
<tr><th colspan="4">SITE CLASSIFICATION</th></tr>
<tr><th>Site Class</th><th>Vs</th><th>N or Nch</th><th>Su</th></tr>
<tr><td>A. Hard rock</td><td>> 5,000 ft/s</td><td>NA</td><td>NA</td></tr>
<tr><td>B. Rock</td><td>2,500 to 5,000 ft/s</td><td>NA</td><td>NA</td></tr>
<tr><td>C. Very dense soil and soft rock</td><td>1,200 to 2,500 ft/s</td><td>> 50</td><td>> 2,000 psf</td></tr>
<tr><td>D. Stiff soil</td><td>600 to 1,200 ft/s</td><td>> 50</td><td>> 2,000 psf</td></tr>
<tr><td rowspan="2">E. Soft clay soil</td><td>< 600 ft/s</td><td>< 15</td><td>< 1,000 psf</td></tr>
<tr><td colspan="3">Any profile with more than 10 ft of soil having the following characteristics:
Plasticity index PI > 20
Moisture content w ≥ 40%, and
Undrained shear strength s_u < 500 psf</td></tr>
<tr><td>F. Soils requiring site response analysis.</td><td colspan="3">Liquefiable soil, highly sensitive clays, collapsible soil
Peat and/or highly organic clays (H > 10 ft of peat or CH, OH)
Very high plasticity clays (H > 25 ft. with PI > 75)
Very thick, soft/medium stiff clays (H > 120 ft with s_u < 1,000 psf.</td></tr>
</table>

Where:

Vs = average shear wave velocity at small shear strains in top 100 ft.
N = standard penetration resistance, ASTM 1586
N_{ch} = average standard penetration resistance for cohesionless soil layer for the top 100 ft.
s_u = average undrained shear strength in top 100 ft.
S_S = 150% g = 1.5 g
S_1 = 60% g = 0.6 g
PGA = 0.34 g (modified from 0.24 g, 10% probability of exceedance in 50 years to 7% probability of exceedance 75 years).
Fpga = Site factor at zero-period on Acceleration spectrum
Fa = Site factor for short period range of acceleration spectrum
Fv = Site factor for long-period range of acceleration spectrum

Site Factors Fpga, $_{Fa,}$ Fv for various periods on Response Spectrum for Guam Bridges			
Site Class	**Fpga** for zero period range	**Fa** for short period range	**Fv** For long period range
A	0.8	0.8	0.8
B	1.0	1.0	1.0
C	1.06	1.0	1.3
D	1.16	1.0	1.5
E	1.08	0.9	2.4
F	Use site specific procedure dynamic response analysis (AASHTO Sec. 3.10.2.2.)		

Use peak ground acceleration, site factors and spectral acceleration values shown above to calculate the design response spectrum shown below.

A_S = Fpga • PGA = Fpga (0.34 g)
S_{DS} = Fa Ss = Fa (1.50 g)
S_{D1} = Fv S1 = Fv (0.60 g)

For periods less than T_0, the elastic seismic coefficient for the mth mode of vibration is:

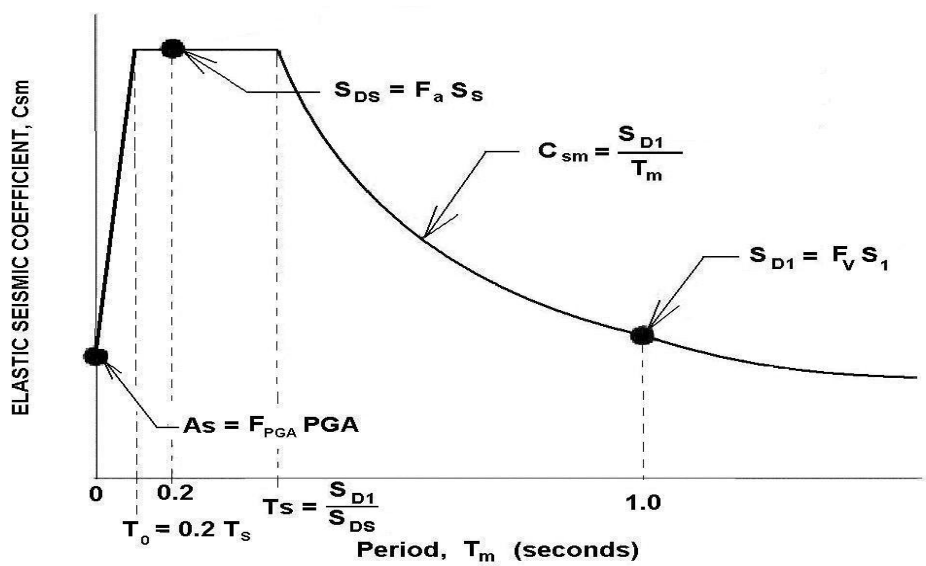
$$C_{SM} = A_S + (S_{DS} - A_S)\,(T_M / T_0)$$

For periods greater than or equal to T_0 and less than or equal to $T_{S,}$ the elastic seismic response coefficient shall be taken as:

$$C_{SM} = S_{DS}$$

For periods greater than T_s, the elastic seismic response coefficient shall be taken as:

$C_{SM} = S_{D1} / T_{M,}$ where, $S_{D1} = F_V S_1 = F_V (0.60\ g)$

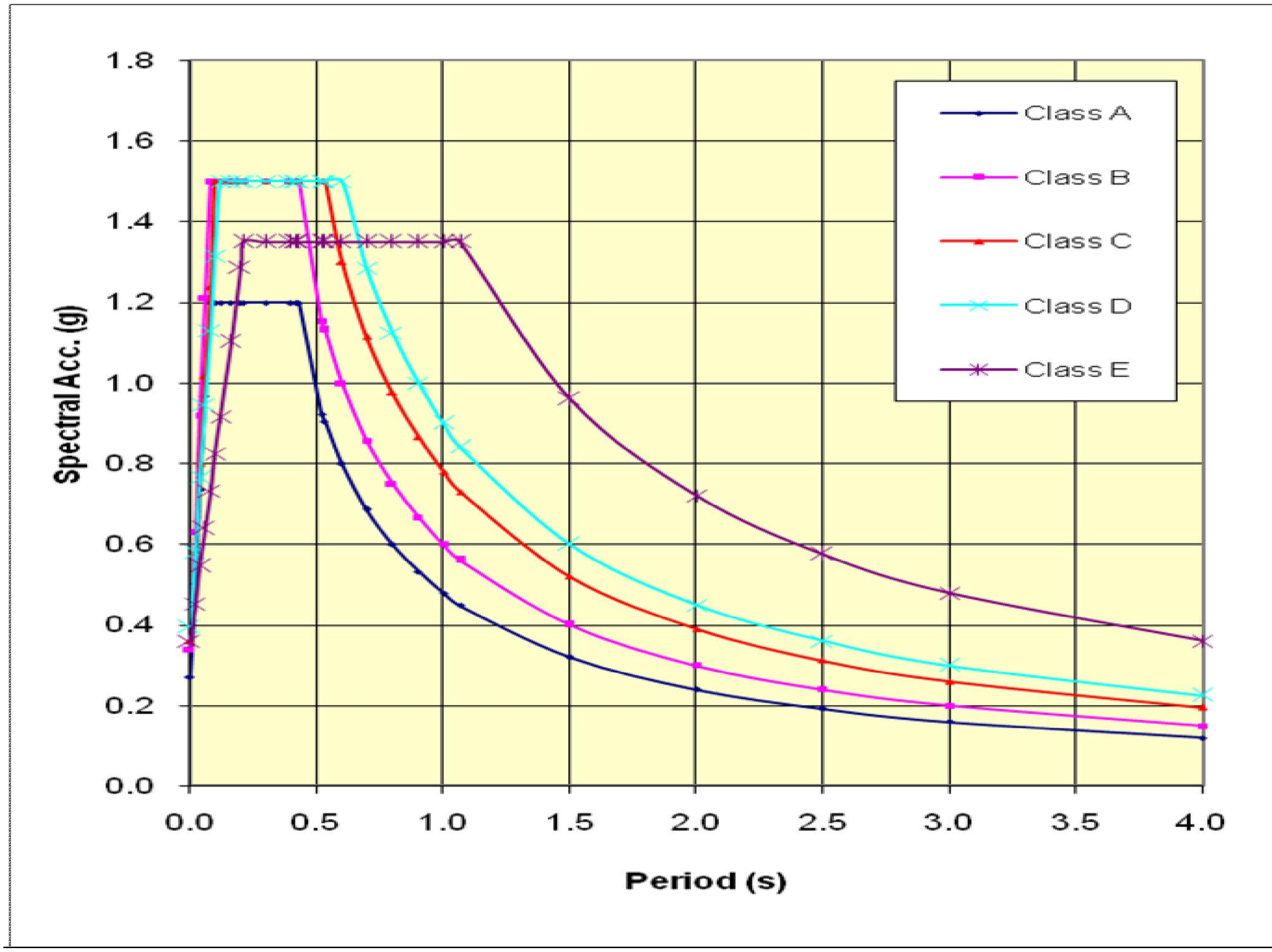


DESIGN RESPONSE SPECTRUM

Response modification factors for substructures and connections shown on Tables 3.10.7.1-1 and 3.10.7.1-2 of AASHTO LRFD (2008 Interim Revisions) shall be used.

Period (sec)	Spectral Accelerations (g)				
	Class A	Class B	Class C	Class D	Class E
0.000	0.2720	0.3400	0.3604	0.3944	0.3672
0.020	0.5040	0.6300	0.5796	0.5787	0.4593
0.040	0.7360	0.9200	0.7987	0.7629	0.5515
0.060	0.9680	1.2100	1.0179	0.9472	0.6436
0.080	1.2000	1.5000	1.2370	1.1315	0.7358
0.100	1.2000	1.5000	1.5000	1.3157	0.8279
0.120	1.2000	1.5000	1.5000	1.5000	0.9200
0.160	1.2000	1.5000	1.5000	1.5000	1.1043
0.210	1.2000	1.5000	1.5000	1.5000	1.3500
0.300	1.2000	1.5000	1.5000	1.5000	1.3500
0.400	1.2000	1.5000	1.5000	1.5000	1.3500
0.520	0.9231	1.1538	1.5000	1.5000	1.3500
0.600	0.8000	1.0000	1.3000	1.5000	1.3500
0.700	0.6857	0.8571	1.1143	1.2857	1.3500
0.800	0.6000	0.7500	0.9750	1.1250	1.3500
0.900	0.5333	0.6667	0.8667	1.0000	1.3500
1.000	0.4800	0.6000	0.7800	0.9000	1.3500
1.070	0.4486	0.5607	0.7290	0.8411	1.3500
1.500	0.3200	0.4000	0.5200	0.6000	0.9600
2.000	0.2400	0.3000	0.3900	0.4500	0.7200
2.500	0.1920	0.2400	0.3120	0.3600	0.5760
3.000	0.1600	0.2000	0.2600	0.3000	0.4800
4.000	0.1200	0.1500	0.1950	0.2250	0.3600

Response spectra table for various soil types for Guam Bridges



Response spectra for various soil types for Guam Bridges

All new bridges shall be designed for a 1,000-year return period (7% probability of exceedance in 75 years).

Bridges with design-year ADT greater than 1,000 vehicles per day, and for which a detour around the bridge would exceed 10 miles shall be designed using importance category "Critical".

Bridges with design-year ADT greater than 1,000 vehicles per day, and for which a detour around the bridge would be less than 10 miles shall be designed using importance category "Essential".

Bridges with design-year ADT less than 1,000 vehicles per day shall be designed using importance category "Other" regardless of detour length.

Response modification factors shown on AASHTO LRFD Tables 3.10.7.1-1 and 3.10.7.1-2 shall be used.

2.1.2.5 Typhoon

In addition to designing for water loads and stream pressure, prescribed in the *AASHTO LRFD Bridge Design Specifications*, account for typhoon wave forces on the bridge per AASHTO publication "*Guide Specifications for Bridges Vulnerable to Coastal Storms".* In many cases, it may be impractical to raise the bridge superstructure above the 100-year design storm elevation. For these situations, provide partial or complete force accommodation per Section 4.4 of the *Guide Specifications for Bridges Vulnerable to Coastal Storms.*

2.1.2.6 Approach Slabs

Approach slabs shall be used for all bridges to accommodate settlement of approach embankments and to accommodate erosion behind abutments that may result from typhoon storm surges. Determine the length of the approach slab using the following formula:

$L = [1.5(H + h + 1.5)] \div \cos\theta \leq 30$ ft

Where: L = Length of the approach slab measured along the centerline of the roadway rounded up to the nearest 5 ft

H = Height of the embankment measured from the bottom of the footing to the bottom of the approach slab (ft)

h = Width of the footing heel (ft)

θ = Skew angle

For coastal bridges, the approach slab shall be at least 25 feet long. The approach slab shall be designed to support the dead load of the approach slab plus live load, with the design span taken to be 2/3 of the total length of the approach slab. Reinforcement determined to be required by the analysis shall be provided for the entire length of the approach slab. Information in the table below may be used in lieu of detailed site specific design for the approach slab if conditions conform to the parameters of the table and notes. Round up the next higher effective span length, do not interpolate.

APPROACH SLAB DESIGN AID									
Length	Thickness	Main Longitudinal Bars				Transverse Bars			
		Top Bars		Bottom Bars		Top Bars		Bottom Bars	
		Size	Spa. (in)	Size	Spa (in)	Size	Spa (in)	Size	Spa (in)
15’	12"	#5	18	#10	10.0	#5	18	#5	9.00
20’	13"	#5	18	#10	7.50	#5	18	#5	8.00
25'	15"	#5	18	#10	7.00	#5	18	#5	8.00
30'	17"	#5	18	#10	6.50	#5	18	#5	8.50

2.1.3 Materials

2.1.3.1 Concrete

Provide concrete in accordance with FP-03 Section 552, modified as appropriate for specific design requirements using SCR 552. Recommended classes and strength of concrete for bridge design are shown in the following table. If the designer proposes to use higher strength concrete than shown in the table, documentation shall be provided to the Department showing that local suppliers can meet the proposed requirements.

RECOMMENDED CONCRETE CLASS AND STRENGTH		
Location	**Class**	**f'$_c$**
Superstructure and Deck	C	4.5 ksi
Barrier and Curb	C	4.5 ksi
Prestressed Beams Release 28 Days	P	4.5 ksi 6.0 ksi
Substructure	A	4.0 ksi
Retaining Walls	A	4.0 ksi
Drilled Shafts	A	4.0 ksi
Precast concrete piles	P	6.0 ksi

2.1.3.2 Reinforcing Steel

Reinforcing steel shall be deformed bars conforming to ASTM Specification A615, A706, or A996, Grade 60.

Minimum reinforcing steel requirements shall conform to AASHTO requirements for shrinkage and temperature reinforcement. Reinforcement for shrinkage and temperature stresses shall be provided near exposed surfaces of walls and slabs not otherwise reinforced.

2.1.3.3 Prestressing Steel

Prestressing steel shall be 0.5 inch nominal diameter, "Uncoated Seven-Wire Low Relaxation Strands for Prestressed Concrete", ASTM A416 Grade 270.

Tensioning force to be applied to each strand to resist design loads shall be 75 percent of ultimate strength or 31.0 kips.

Modulus of elasticity, E = 28,500 ksi is assumed (AASHTO 5.4.4.2).

2.1.3.4 Structural Steel

Types of structural steel to be selected for use in the design and construction of bridges are as follows:

- ASTM A709 grade 50W shall be specified for an un-coated weathering steel bridge
- ASTM A709 grade 50 shall be specified for a coated steel bridge

ASTM A709 grade 36 is not recommended and is being discontinued by most mills.

2.1.3.5 Non-recommended Materials

The use of the following materials, for primary load carrying members of the superstructure and substructure are not recommended due to maintenance and constructability issues. Primary members generally include stringers, cross beams, and columns.

- Structural Steel (for coastal bridges)
- Timber (solid sawn and glue-laminated)
- Fiber reinforced polymer

2.1.4 Bridge Structure Types

Constraints imposed by the location of the island from the major sources of materials, labor and equipment as well as the proximity of main transportation routes to the coastline result in certain types of structures being less desirable. Steel truss bridges and cable bridges (suspended cables or cable stayed types) involve specialized labor for construction that is not currently available on Guam. Maintenance of certain structure types will also strain Guam DPW's maintenance crews. The island would therefore be best served by using structures made of concrete and reinforced with high strength bars and high strength prestressing strands.

Integral bridge construction is highly recommended. Integral construction involves attaching the superstructure and substructure (abutment) together. The longitudinal movements are accommodated by the flexibility of the abutments (capped pile abutment on single row of piles regardless of pile type).

Where integral bridge construction is not feasible (such as for foundations on spread footings), semi-integral construction details should be designed.

Skews greater than 40 degrees are strongly discouraged. For precast, prestressed box beam bridges, skews greater than 30 degrees are prohibited.

2.1.4.1 Recommended Bridge Superstructure Types

The following are some of the bridge superstructure types that are economical to construct and maintain on the island:

- Precast, Prestressed AASHTO Girders
- Precast, Prestressed Box Beams
- Precast, Prestressed Tee Beams
- Reinforced Concrete Deck Girders
- Reinforced Concrete Slab bridges
- Three-Sided Precast Concrete Culverts

2.1.4.2 Non-recommended Bridge Superstructure Types

The use of the following bridge types are not recommended due to maintenance issues or observed problems on similar types on the island.

- Structural Steel Trusses
- Structural steel beam or girders for coastal bridges
- Precast or prestressed tees or double tees with webs less than 10 inches thick and flanges less than 6 inches thick.
- Other structures where the primary load carrying members will require corrosion monitoring or anti-corrosion maintenance.

2.1.4.3 Requirements for Steel Beam and Girder Bridges

For steel beam or built-up girder bridges provide a camber tabulation table on a structural steel detail sheet. Tabulation is required regardless of the amount of deflection and is required for all beams or girders, if the deflection is different. The table is to include bearing points, quarter points, center of span, splice points, and maximum 30 foot increments. Unique geometry may require an even closer spacing.

When designing curved steel girder structures, investigate all temporary and permanent loading conditions including loading from wet concrete in the deck pour for all stages of construction. Consider future re-decking as a separate loading condition. Design diaphragms as full load carrying members. The Designer shall perform a three-dimensional analysis representing the structure as a whole and as it will exist during all intermediate stages and under all construction loadings. Such analysis is essential to accurately predict stresses and deflections in all girders and diaphragms and to ensure that the structure is stable during all construction stages and loading conditions.

The Designer shall supply basic erection data on the contract plans. As a minimum, include the following information:

- If temporary supports are required, provide the location of the assumed temporary support points, reactions and deflections for each construction stage and loading condition.
- Instructions to the Contractor as to when and how to fasten connections for cross frames or diaphragms to assure stability during all temporary conditions.

Further design information for curved structures is contained in the "Guide Specifications for Horizontally Curved Highway Bridges", published by the American Association of State Highway and Transportation Officials.

2.1.4.4 Requirements for Prestressed Concrete Superstructures

Model multi-span, non-composite members as simple-span for all loading conditions. Model multi-span, composite members using the two loading conditions that follow. The loading condition that produces the largest load effects shall govern.

- Simple-span for non-composite dead loads; continuous span for live load and composite dead loads.
- Simple-span for all loading conditions. Do not include future wearing surface.

Box beams shall be limited to a maximum skew of 30 degrees. Box beams shall be supported by two bearings at each support. Abutment wingwalls above the bridge seat and backwalls should not be cast until after box beams have been erected. The cast in place wingwall and box beam should normally be separated by one inch joint filler. The designer should show both requirements in the plans. Casting the backwall and wingwalls after the box beams are erected eliminates installation problems associated with the actual physical dimensions of the box beam and the joint filler.

Debonding of prestressing strands, by an approved sheath, shall be done as required to control stresses at the ends of beams and girders. The following guidelines shall be followed for debonded strand designs:

- The maximum debonded length at each end shall not be greater than 0.16L – 40 inches. Where L equals the span length in inches.
- A minimum of one-half the number of debonded strands shall have a debonded length equal to one-half times the maximum debonded length.
- No more than 25% of the total number of strands in the I-beam shall be debonded.
- No more than 40% of the strands in any row shall be debonded.
- Debonded strands shall be symmetrical about the centerline of the beam.
- Strands extended from a beam to develop positive moment resistance at pier locations shall not be debonded strands.
- Locate debonded strands as high as possible in the bottom flange to aid in the placement of the sheath during fabrication.
- The designer shall show on the detail plans the number, spacing and the length of required debonding per strand.

Draping or deflecting of strands in box beams is not permitted. Draping of strands in AASHTO girders is a generally considered to be better technique to control tension at the ends of precast girders than debonding, but the draping methodology is not currently available from local suppliers and is therefore not currently permitted. An alternative method is to cast the girder with draped ducts for post tensioning. Designers are encouraged to check with local precasters to ascertain whether draping of strands for AASHTO girders has become available.

2.1.4.5 Decks

It is recommended that only cast-in-place concrete decks be designed and used on Guam. Precast panel alternatives have shown cracking problems at the joints between the panels and there are questions on the transfer of stresses in the finished deck sections.

A deck pour sequence is required for all prestressed I-beam designs made continuous at pier locations. Concrete should be placed within the positive moment regions of the girders prior to placing concrete over the piers.

The concrete deck design shall be in conformance with the approximate elastic methods of analysis specified in the AASHTO LRFD Bridge Design Specifications and the additional requirements specified in this Manual. Refined methods of analysis and the empirical design method, LRFD 9.7.2, are prohibited. The design live load shall be HL-93 and the design dead load shall include an allowance for a future wearing surface equal to 0.03 k/ft^2. Provide clear cover from the top and bottom surface of the deck to the main transverse reinforcing steel as specified in Section 2.1.2.3 of these guidelines. Consider the upper 1 inch of the deck to be a monolithic wearing surface that does not contribute to the structural capacity of the deck or of the composite section.

Deck designs for superstructures with effective span lengths ranging from 7.0 ft. to 14.0 ft. in 0.5 ft. increments are provided in the deck design aid below. These designs apply for the full length of the bridge and preclude the need for additional transverse reinforcement at supported deck ends. Information in the table may be used in lieu of detailed site specific design for the deck if conditions conform to the parameters of the table and notes. Round up to the next higher effective span length do not interpolate.

CONCRETE DECK DESIGN AID												
Eff. Span Length (ft.)	Deck Thickness (in.)	Overhang Deck Thickness (in.)	Main Transverse Steel						Longitudinal Steel			
			Top Bars				Bottom Bars		Top Bars		Bottom Bars	
			Size	Spa. (in.)	Additional Overhang Bar Size	Cutoff Length (in.)	Size	Spa. (in.)	Size	Spa. (in.)	Size	Spa. (in.)
7.0	8.50	10.50	#5	6.50	#5	50	#5	6.50	#4	12.50	#5	13.00
7.5	8.50	10.50	#5	6.25	#5	50	#5	6.25	#4	12.00	#5	12.25
8.0	8.50	10.50	#5	6.00	#5	50	#5	6.00	#4	11.50	#5	11.5
8.5	8.75	10.75	#5	6.00	#4	50	#5	6.00	#4	11.50	#5	11.25
9.0	8.75	10.75	#5	5.75	#4	50	#5	5.75	#4	11.00	#5	10.75
9.5	9.00	11.00	#5	5.75	#4	50	#5	5.75	#4	11.00	#5	10.50
10.0	9..25	11.25	#5	5.50	#4	50	#5	5.50	#4	10.50	#5	10.50
10.5	9.25	11.25	#5	5.25	#4	50	#5	5.25	#4	10.00	#5	10.00
11.0	9.50	11.50	#5	5.00	#4	38	#5	5.00	#4	9.50	#5	10.00
11.5	9.75	11.75	#5	5.00	#4	38	#5	5.00	#4	9.50	#5	9.75
12.0	9.75	11.75	#6	6.00	#4	38	#5	6.00	#4	8.00	#5	9.50
12.5	10.00	12.00	#6	6.00	#4	21	#5	6.00	#4	8.00	#5	9.25
13.0	10.25	12.25	#6	6.00	#4	21	#5	6.00	#4	8.00	#5	9.25
13.5	10.25	12.25	#6	5.75	#4	21	#5	5.75	#4	7.75	#5	9.00
14.0	10.50	12.50	#6	5.75	#4	21	#5	5.75	#4	7.75	#5	9.00

Notes for Concrete Deck Design Aid Table:

- Design is in accordance with AASHTO LRFD Bridge Design Specifications
- Design Assumptions:
  - ◊ Four or more beam/girder lines
  - ◊ Transverse steel is placed perpendicular to beam/girder lines
  - ◊ Normal weight concrete with f′c = 4.5 ksi
  - ◊ Reinforcing steel with fy = 60 ksi
  - ◊ Monolithic Wearing Surface = 1.0 in.
  - ◊ Future Wearing Surface = 0.03 ksf
  - ◊ LRFD 5.7.3.4 - Exposure Factor (γe) = 0.75
  - ◊ Top cover = 2.50 in.; Bottom cover = 1.50 in.
  - ◊ Maximum overhang = 4.0 ft. (measured from centerline of fascia beam/girder to deck edge).

- Calculate Effective Span Length according to LRFD 9.7.3.2 and round up to the nearest 0.5 ft. increment.
- Cutoff Length = length beyond the centerline of the fascia beam/girder where additional overhang bars are no longer required for strength.
- Longitudinal bar spacing does not include the additional reinforcing steel required for negative moments in accordance with LRFD 5.7.3.2 (for prestressed beams) and LRFD 6.10.1.7 (for steel beams/girders). Add additional longitudinal reinforcing steel in negative moment regions for continuous bridges as required.

2.1.4.6 Recommended Bridge Substructure Types

The following are bridge substructure types that are appropriate for use on Guam

- Deep Foundations (driven piles and drilled shafts)
- Spread Footings founded on bedrock

Substructure footing elevations should be shown on the Final Structure Site Plan. The top of footing should be a minimum of one foot below the finished ground line. The top of footing should be at least one foot below the bottom of any adjacent drainage ditch. The bottom of footing shall not be less than three feet below and measured normal to the finished ground line.

Preference should be given to the use of integral spill-thru type abutments. Generally for integral stub abutments on piling or drilled shafts the shortest distance from the surface of the embankment to the bottom of the toe of the footing should be at least 3'-0". For stub abutments on spread footing on soil, the minimum dimension shall be 5'-0". Wall type abutments should be used only where site conditions dictate their use.

For waterway bridges, the following pier types should be used:

- Capped pile type piers; generally limited to an unsupported pile length of 20 feet. For unsupported pile lengths greater than 15 feet, the designer should analyze the piles as columns above ground. Scour depths and the embedded depth to fixity of the driven piles shall be included in the determination of unsupported length.
- Cap-and-column type piers.

For highway grade separations, the pier type should generally be cap-and-column piers supported on a minimum of 3 columns. The purpose for this provision is to reduce the potential for total pier failure in the event of an impact involving a large vehicle or its cargo. This requirement may be waived for temporary (phased construction) conditions that require caps supported on less than 3 columns. Typically the pier cap ends should be cantilevered and have squared ends.

2.1.4.7 Spread Footings

Spread footings are not recommended for use on Guam unless the foundation is constructed on bedrock. Spread footings are prohibited for use on Guam for stream crossings and coastal bridges unless the foundation is constructed on bedrock. The use of spread footings shall be based on an assessment of the following: design loads; depth of suitable bearing materials; ease of construction; effects of flooding and scour analysis; liquefaction and swelling potential of the soils, and amount of predicted settlement versus tolerable structure movement.

Spread footings shall be designed in accordance with LRFD 10.6.

Elevations for the bottom of the footing shall be shown on the Final Structure Site Plan. The estimated size of the footing; estimated settlements; and the factored bearing resistances shall be provided for review with the Foundation Report.

Adjust the footing size, the amount of predicted settlement and the factored bearing resistance during detail design as the design loads for the Service, Strength and Extreme Event Limit State are refined.

All spread footings at all substructure units, not founded on bedrock, are to have elevation reference monuments constructed in the footings. This is for the purpose of measuring footing elevations during and after construction for the purpose of documenting the performance of the spread footings, both short term and long term.

2.1.4.8 Deep Foundations

Pile foundations should be considered when spread footing foundations are prohibited or are not feasible.

The type, size and estimated length of the piles for each substructure unit shall be shown on the Final Structure Site Plan. The estimated length for piling shall be measured from pile tip the cutoff elevation in the pile cap and shall be rounded up to the nearest five feet.

Piles should be precast, prestressed concrete. Common shapes and sizes of concrete piles available on Guam include 10", 12", 14", 15", 16", 18", 20", and 24" square. The designer must confirm availability with local producers if a size or shape different from those listed is proposed for use in foundations.

For piles driven to refusal on bedrock, refusal is met when the pile penetration is an inch or less after receiving at least 20 blows from the pile hammer. Verify pile tip elevation with the nearest soil boring log to insure resistance to driving is not due to isolated rock or pile obstruction.

Piles not driven to refusal on bedrock develop their geotechnical resistance by a combination of soil friction or adhesion along the sides of the pile and end bearing on the pile tip.

When a pile must resist uplift loads, the uplift resistance shall be calculated in accordance with LRFD 10.7.3.10. Use static analysis methods (LRFD 10.7.3.8.6) to determine the nominal uplift resistance due to side resistance. Where the estimated pile length is controlled by the required uplift resistance, specify a minimum penetration pile tip elevation.

Drilled shafts should be considered when their use would:

- Prevent the need of cofferdams.
- Become economically viable due to high design loads (eliminates the need of large quantities of pile).
- Provide protection against scour.
- Provide resistance against lateral and uplift loads.
- Accommodate sites where the depth to bedrock is too short for adequate pile embedment but too deep for spread footings.
- Accommodate the site concerns associated with pile driving process (vibrations, interference due to battered piles, etc.).

Drilled shafts shall be designed in accordance with LRFD 10.8. Additional design guidance can be obtained from the report '*Drilled Shafts: Construction Procedures and LRFD Design Methods*,' FHWA Geotechnical Engineering Circular (GEC) 010 available from FHWA at the following web site:

http://www.fhwa.dot.gov/engineering/geotech/foundations/nhi10016/nhi10016.pdf

Drilled shafts that support pier columns shall be at least 6 inches larger in diameter than the pier column diameter. The minimum diameter for drilled shafts that support pier columns shall be 42 inches. The minimum diameter for all other drilled shafts shall be 36 inches. Drilled shaft diameters of less than 36 inches [915 mm] are not recommended.

Under-reams or belled shafts should not be used. Belled shafts are difficult to construct under water or slurry and the bell will collapse in non-cohesive soils. Cleaning and inspecting the base of the drilled shaft within the bell are also very difficult.

Drilled shaft diameters shall be shown on the Final Structure Site Plan. For drilled shafts with friction type design, the tip elevation shall also be shown. For drilled shafts supported on bedrock, the tip elevation should not be given. Instead, the approximate top of the bedrock elevation and the length of the bedrock socket shall be shown in the profile view on the Final Structure Site Plan. Designers should neglect the contribution to skin friction provided by the top 2 ft. of the rock socket.

Designers shall indicate on plans that drilled shaft integrity will be verified with a pile data analyzer (PDA) using crosshole sonic logging (CSL), gamma ray logging or crosshole tomograhic (CT) techniques to determine defects in the drilled shaft. Size and number of pipes embedded in drilled shaft, for integrity testing, shall be shown on plans.

2.2 Construction Cost Estimating

Develop a construction cost estimate for each major phase of the project. Include mobilization and construction survey and staking as line items in the estimate. Typically mobilization is 10% of the cost of the construction items and construction survey and staking is 3% of the cost of the construction items. Pay items numbers, units of measure, unit prices for material labor and equipment and quantities shall be shown on the estimating sheet.

See Appendix B for a cost estimate sample and NCHRP Report 483 on life cycle cost estimating.

2.2.1 Preliminary Engineering Estimate

During Preliminary Engineering, develop a life cycle cost analysis for each alternative considered. Construction cost estimates should be based on the estimated quantities and unit costs for the major high cost categories of work and a percentage of total construction costs for minor categories of work. Unit costs should be based on recent bid experience and should be adjusted as appropriate to reflect unique requirements of the project. A substantial contingency factor should be added to preliminary cost estimates to account for elements that have not been fully developed, typically 30%. Useful information for life cycle cost estimating can be obtained from FHWA at the following web site:

http://www.fhwa.dot.gov/publications/publicroads/05nov/09.cfm

Consider the following major items for development of estimated quantities and unit costs:

- Archaeological and/or environmental costs
- Deck concrete [cubic yard]
- Girders [each or linear foot]
- Abutment concrete [cubic yard]
- Foundations: Piling, Drilled Shafts [linear foot]
- Bridge railing [linear foot]

Miscellaneous minor items may be grouped into categories as a lump sum or percentage of the total construction (include temporary traffic control, guardrail, signing, striping, erosion and sediment control, fences, re-vegetation, landscaping, etc.) based on historical data of similar projects. Incorporate these into a Contingencies line item. Typically, Contingencies for this phase in the project are 25% of the cost of the construction items.

2.2.2 Final Design ("Engineer's") Estimate

During the Final Design phase, develop a detailed construction cost estimate (Engineer's Estimate), listing of all items of work in the contract, showing quantity, unit of measurement, unit cost and total cost of each. Contingencies and other costs added to the construction estimate makes up the project amount. Typically, Contingencies for this phase in the project are 5-10% of the cost of the construction items to account for uncertainties in bid unit prices.

Retain confidentiality of the unit price analysis and Engineer's Estimate at all times to maintain the integrity of the bidding and procurement process.

2.3 Construction Schedule

Determine the anticipated construction schedule including reasonable times for completion of all construction activities. The schedule must show the time required per activity, the remaining time to complete the project and the total construction time. Factors that will affect construction duration such as material availability, traffic restrictions, in-water work windows, weather delays and material import lead times must be taken into consideration.

Construction schedule shall be in Gantt chart format and shall show detailed activities of major task subdivided into multiple sub-task. Various construction scheduling software are available on the market (i.e. Primavera, Microsoft Project, etc) and should be used. The construction schedule must be updated monthly and submitted to DPW or DPW's designated Construction Managers to enable an effective tracking of the project.

A sample construction schedule is shown in Appendix C.

2.4 Preparation of Plans

Drawings should be so planned that all details will fall within the prescribed borderlines. All detail views should be carefully drawn to a scale large enough to be easily read when reduced to half size. Views should not be crowded on the sheet.

The scale of the views on the drawings should not be stated because in making reproductions of the drawing the prints may be either the same size as the drawing or half-size.

A North Arrow symbol should be placed on the Site Plan, General Plan and all plan views.

Elevation views of piers and the forward abutment should be shown looking forward along the stationing of the project. The rear abutment should be viewed in the reverse direction. Rear and forward abutments should be detailed on separate plan sheets for staged construction projects or for other geometric conditions that produce asymmetry between abutments.

When describing directions or locations of various elements of a highway project the centerline of construction (survey) and stationing should be used as a basis for these directions and locations. Elements are located either left or right of the centerline and to the rear and forward with respect to station progression. [e.g. rear abutment; forward pier; left side; right railing; left forward corner]

For structures on a horizontal curve a reference line, usually a chord of the curve shall be provided. This reference line should be shown on the General Plan/Site Plan view with a brief description, including, for example, "Reference Line (centerline bearing to bearing)," and the stations of the points where the reference line intersects the curve. Skews, dimensions of substructure elements and superstructure elements should be given from this Reference Line, both on the General Plan/Site Plan and on the individual detail sheets. Dimensions from the curve generally should be avoided. The distance between the curve and reference line should be dimensioned at the substructure units. In this manner a check is available to the contractor. The reference tangent can be used if appropriate, such as for bridges that are partially in curve and partially in tangent.

For each substructure unit, the skew angle should be shown with respect to the centerline of construction or, for curved structures, to a reference chord. The skew angle is the angle of deviation of the substructure unit from perpendicular to the centerline of construction or reference chord. The angle shall be measured from the centerline of construction or reference chord to a line perpendicular to the centerline of the substructure unit or from a line perpendicular to the centerline of construction or reference chord to the centerline of the substructure unit.

In placing dimensions on the drawings, sufficient overall dimensions will be given so that it will not be necessary for a person reading the drawings to add up dimensions in order to determine the length, width or height of an abutment, pier or other element of a structure.

In general, the designer should avoid showing a detail or dimension in more than one place on the plans. Such duplication is usually unnecessary and always increases the risk of errors, particularly where revisions are made at a later date.

If, because of lack of space on a particular sheet, it is necessary to place a view or a section on another sheet, both sheets should be clearly cross-referenced.

Abbreviation of words generally should be avoided. Abbreviations, unless they are in common use, may cause delay and uncertainty in interpreting the drawings. If abbreviations are used, a legend should be provided to explain the abbreviation.

Plan sheet size to be used is 11" x 17". Margins shall be 1" on the left edge and 1/4" on all other edges.

Where a project includes more than one bridge, plan preparation economies may be obtained by coordination of the individual plans. Where general notes are numerous and extensive, time can be saved by using a sheet of notes common to all bridges, or by including all of the common notes on one bridge plan and referring to them on the other bridge plans. The same applies to common details.

A set of completed bridge plans should conform to the following order:

- Site Plan
- General Plan & General Notes
- Estimated Quantities & Phase Construction Details
- Abutments
- Piers
- Superstructure
- Railing Details
- Expansion device details
- Approach slab details

The General Plan sheet does not require an elevation view. The General Plan sheet is only required for:

- New bridge of variable width or curved alignment
- New bridge requiring staged construction

If no General Plan sheet is furnished, the bridge plans may require a line diagram to show stationing and bridge layout dimensions that would not be practical to show on the site plan due to the site plan's scale. Other details may be required to adequately present information needed to construct the bridge.

2.4.1 Electronic File Format

Construction drawings must be submitted to DPW prepared using MicroStation software (file extension =.DGN). Drawings prepared in AutoCAD software must be converted to MicroStation file.

Unless noted otherwise, CAD drafting conventions and standards shall follow the guidelines published by the Central Federal Lands division of the Federal Highway Administration. CAD drafting standards can be obtained from:

http://www.cflhd.gov/resources/CADD/

2.4.2 Scale and Units

All CAD drawing shall be drafted at full scale in architectural units (one drawing unit equals one inch).

2.4.3 Tolerances

Bridge element details recorded within CAD drawings must reconcile within ½ of actual field dimensions as measured in the field and overall bridge plans and elevations must reconcile to within one inch of the actual bridge dimension. Notify DPW field representative of any dimension error tolerances on the project.

2.4.4 Title Blocks

Title blocks for construction drawings must use the Guam Department of Public Works block template. Title blocks shall contain all of the information listed below:

Project Information:

- Firm Name – representing the drawing author
- Project Name – as specified by Guam Department of Public Works
- Project Number – as specified by Guam Department of Public Works

Drawing Information:

- Drawing Title – Indicating the drawing content. e.g. bridge plan, elevation, section, detail, etc.
- Drawing Number
- Date of Drawing – original drawing date including significant revision dates.
- Drawing Scale – representing the plot scale of the drawing with the title block.
- North Arrow

2.4.5 External Reference Files (XREF's)

Guam Department of Public Works will not accept the submission of any CAD drawing deliverable which contains references to external source drawing files. All externally referenced data sources that were used during the CAD drawing production phase shall be inserted and retained as a block within a single drawing file, including the title block, upon project completion and prior to drawing delivery to DPW.

2.5 Standard Drawings

Use Guam Standard Drawings when available. When significant changes to a standard drawing are needed, combine applicable Standard Drawing details with modified details in a project specific drawing. All drawings to be prepared must bear a graphic scale at the lower right hand corner of the sheet to enable scaling of the plans and details to any size construction plans are needed for the project.

2.6 Construction Specifications

The following specifications, including current revisions, apply to all bridge projects:

- *Standard Specifications for Construction of Roads and Bridges on Federal Highway Projects*, FP-03, U.S. Customary Units, Publication No. FHWA-FLH-03-002.
- Special Contract Requirements (SCRs) prepared for the specific project.
- *AASHTO LRFD Bridge Construction Specifications*, American Association of State Highway and Transportation Officials (AASHTO), Customary U.S. Units, current edition with interim revisions.

3.0 PRELIMINARY ENGINEERING

The preliminary engineering process consists of evaluating feasible bridge concepts, selecting a preferred alternative, and assembling the findings into the Bridge Type, Size and Location (TS&L) Package, which is provided to DPW and key stakeholders for their review, comment and acceptance.

FHWA promotes the use of accelerated bridge construction technology to reduce onsite construction time. Use of prefabricated bridge elements is a common method to achieve accelerated bridge construction. Useful information on accelerated bridge construction and prefabricated bridge elements can be downloaded from FHWA at the following web sites:

http://www.fhwa.dot.gov/bridge/abc/index.cfm
http://www.fhwa.dot.gov/bridge/prefab/if09010/report.pdf

It is the responsibility of the designer to determine the feasibility and suitability of the various accelerated bridge methodologies proposed for a particular design, considering local contractor and supplier limitations, and special requirements for seismic forces and coastal storm events.

DPW acceptance of a bridge project's TS&L is a critical point of the decision making process that establishes the geometric boundaries of the project footprint and allows for concurrent right-of-way, environmental permitting and construction contract document activities to proceed.

The Bridge TS&L package shall include Draft and Final versions of the Bridge TS&L Report. The Report includes appendices containing the following:

- Bridge Plan and Elevation Drawing for the recommended preferred alternative
- Engineer's Preliminary Cost Estimate for each feasible alternative considered
- Construction Schedule for the preferred alternative
- Identification of Design Exception Request(s)
- Foundation (geotechnical) report and recommendations
- Hydraulics Report

3.1 Start-Up Tasks

3.1.1 Data Collection and Literature Review

Structure Data Collection
The Engineer should review existing structure information (as available), such as the "As Built" drawings, maintenance records and the most recent Inspection Reports, to become familiar with the project site and the existing structure.

Hydraulic Data Collection
Collect and review existing data including such items as flood photos; aerial photos; topographic maps; Federal Emergency Management Agency ("FEMA") reports; maps, and hydraulic model; tsunami and storm surge reports; existing hydraulic files data; and scour information.

Multiple methods may be used to provide adequate confidence data, for the predicted discharge, which includes FEMA flood insurance studies, Corps of Engineers flood studies, United States Geological Survey ("USGS") gauging stations, regression equations, and TR-55 hydrographs. Evaluate floodplain, deck drainage, temporary water management, and sediment control impacts and measures for construction purposes.

Other Data Collection
Collect and review other existing data such as aerial photos, topographic maps, existing geotechnical data reports, site geologic maps, previous geotechnical investigations, the digital terrain model (DTM), Utility maps and right-of-way information.

3.1.2 Site Visit

After the literature review, a site visit should be performed, to document the condition of the existing structures and project site characteristics (including photographs). The information from the literature review will generally be confirmed by the site visit. Any discrepancies shall be brought to the attention of DPW. Copies of all inspection reports must be submitted to DPW for their files and action.

Information gathered in the Startup Tasks will be incorporated, as appropriate, in the preliminary engineering effort.

3.1.3 Subsurface Exploration

Subsurface Exploration Work Plan
Borings will require acquisition of a permit from DPW, and the permit will require Section 106 clearance from SHPO. Allow at least 30 calendar days in the design schedule for permit approval. Take sufficient borings at the project site to determine the recommended foundation type. This generally requires one boring in the vicinity of each proposed pier and abutment location to a depth necessary to reach bedrock, but not more than 120 feet. Consider (where applicable) the effects of settlement, fills, surcharges, lateral spreading, seismic loading, soil liquefaction and scour. Coordinate the geotechnical design work and the geotechnical investigation work.

Prepare and submit a Subsurface Exploration Work Plan which describes the details of the work to be performed at the project site during the subsurface exploration. Include in this plan, details describing geotechnical activities to be addressed at the project site, including access; environmental permitting; subsurface exploration means and methods; site restoration; traffic control; and health & safety. The Work Plan must consider geologic, seismic, and groundwater conditions; potential foundation types and construction methods; potential field testing; and pavement design needs at the Project.

Subsurface Exploration Work Plan Implementation
Perform a subsurface exploration of the proposed project site as described in the Subsurface Exploration Work Plan following receipt of necessary permits and utility clearance.

3.2 Bridge TS&L Report

The Bridge TS&L Report summarizes the preliminary engineering that is used to develop the recommended concept design and provides enough information so that DPW and key stakeholders can effectively evaluate the proposed concept.

3.2.1 Draft TS&L Report

Provide a Draft TS&L Report, which includes an alternatives analysis for different bridge types investigated, configurations considered and a complete discussion of the preferred alternative, as well as the controlling factors used to arrive at the recommendation.

Organize the Report so that it will follow the format described below.

1. Project Description: Provide a description of the general Project background and the reason for the Project.
2. Roadway and Bridge Geometrics: Include a description of the roadway approaches to the bridge, which will include items such as the vertical and horizontal curve data, cross slope, number of lanes, shoulder width, ADT, sidewalks or other pedestrian facilities and general terrain conditions around the bridge. In the description of the bridge geometry, note any differences in shoulder widths, tapers, medians, sidewalks or other items which may be unique or differ from the roadway section.
3. Utilities: Provide the name, location and disposition of all of the utilities in the area of the bridge.
4. Right-of-Way: Discuss available right-of-way and any restrictions present or access issues.
5. Environmental Considerations: Discuss any applicable environmental permits and restrictions. Discuss the presence of any hazardous material, such as lead based paint or asbestos. Discuss any pertinent archeological issues or endangered species. Discuss bridge aesthetics and compatibility with surroundings as applicable.
6. Geotechnical: Include a brief summary of the Foundation Report and the recommendations of substructure types. Soil resistivity tests shall be performed where steel piles or steel structures and cathodic protection method of corrosion mitigation is considered. Boring logs and laboratory test results shall be included in the Appendix to the report.
7. Hydraulics: Include a brief summary of the Hydraulics Report and the recommendations. Include a discussion of any scour issues and mitigation as applicable. Channel cross sections at the bridge crossing indicating the flood levels in relation to the new bridge profile must be shown in the Hydraulics Report.
8. Design Criteria: Provide a complete list of all design guidelines and design criteria, which will be used to design the bridge. Criteria should include applicable loading, such as live loading (including military vehicles as applicable), typhoon and seismic loading.
9. Construction: Consider and discuss the following construction factors:

   - Access to site and available staging areas
   - Construction duration
   - Detours or stage construction required
   - Potential erection problems
   - Ease or difficulty of construction

10. Safety: Consider and discuss safety issues for the project, such as:

    - Traffic density and speed
    - Construction impact to the safety of the travelling public
    - Approach guardrail type and connection to the bridge
    - Bridge Rail type

11. Structure Alternatives Considered: Discuss the structure alternatives considered. Discuss how issues such as whether a bridge or a culvert, new bridge versus widening the existing bridge and a new bridge versus a repair of the existing bridge were resolved. Factors to be considered with regard to the structural aspects of a project include:

    - Life cycle costs
    - Span-Depth ratios
    - Horizontal and vertical clearances
    - Limitations on structural depth
    - Future widening ability
    - End slope treatment
    - Foundation and groundwater conditions
    - Anticipated settlement
    - Eliminate deck joints by use of integral or semi-integral construction

12. Description of Recommendation: Provide a description of the recommended structure. Discuss all pertinent topics such as:

    - The substructure type and the depth of piles or drilled shafts or the size of spread footing given.
    - The final span arrangement along with any other alternative arrangements considered and why they were not selected as the preferred alternative for the Project.
    - The configuration of the bents and estimated column sizes and locations, the size of the cap beam and the location of girder lines.
    - The preferred superstructure type and typical section.
    - Conceptual Cost Estimate: The level of bridge design will be adequate to make a cost comparison between the different types of structures and layouts.
    - Stage construction scheme and detour requirements.
    - Proposed bridge railing.
    - Bridge deck drainage requirements.

Describe the recommended preferred alternative design and summarize the key items that led to the recommendation. Economy is generally the best justification for a selection. However, a life cycle cost analysis, as well as some of the above considerations may outweigh differences in initial project cost.

In the final analysis, DPW and key stakeholders must be satisfied that the proper bridge has been selected.

3.2.2 Draft TS&L Report Appendices

Bridge Plan and Elevation Drawing
As part of the Bridge TS&L Report appendices, the recommended alternative is to be shown in a TS&L Plan and Elevation drawing, produced on 11" X 17" paper print. The TS&L Plan and Elevation sheet is normally drawn on one sheet to a 1"=20' scale for smaller structures and 1"=40' scale for larger structures.

Generally, this drawing contains a Plan, Elevation, Section and Stage Construction diagram for the recommended structure and shows details such as:

- The substructure type and the depth of piles or drilled shafts or the size of spread footings.
- The final span arrangement with the span numbers.
- The configuration of the bents with the bent numbers.
- The estimated column sizes and locations.
- The size of the cap beam and the location of girder lines
- The proposed bridge railing
- The preferred superstructure type and typical section
- A construction staging scheme
- All Right of Way limits will be clearly shown on the plan views of the bridge

Preliminary Cost Estimate for Bridge Construction
Develop a construction cost estimate for each alternative considered. Place preliminary cost estimates for each bridge alternative, and back up for the unit costs including quotes from suppliers if applicable, in the appendices of the Bridge TS&L Report.

Construction Schedule
Develop a construction schedule that depicts the estimated construction sequence duration by phase for the recommended preferred alternative.

Identification of Design Exceptions
If preliminary engineering concludes that achieving normal design criteria is not practical, evaluate the consequences and document each decision for exception to the standards in a technical memorandum. Identify and discuss existing substandard conditions or elements that are not reconstructed to approved, current standards as part of the project.

Guidance on evaluating design exceptions can be downloaded from the following FHWA web site:

http://www.fhwa.dot.gov/design/0625sup.cfm

3.2.3 Final TS&L Report

Update and revise the Draft TS&L Report; incorporate review comments received from DPW and key stakeholders and deliver the sealed Final TS&L Report to the Department.

3.3 Foundation Report and Data Sheets

3.3.1 Draft Foundation Report

Prepare a Draft Foundation Report that includes presentation of:

- Results of subsurface exploration and laboratory testing
- Site geologic and seismic setting; and recommendations and design parameters for deep and shallow foundations (piles, drilled shafts, footings) including potential for downdrag resulting from liquefaction
- Temporary shoring; retaining walls (conventional & Mechanically Stabilized Earth ["MSE"]),embankment fills
- Excavations
- Pavement subgrades and drainage conditions
- Pertinent geologic hazard mitigation recommendations
- Soil resistivity test results, if steel piles/sheet piles will require cathodic protection
- Recommended locations of drive test piles and load test piles, if required

General guidance for the foundation report can be found in Appendix E. Additional guidance pertaining to seismic analysis and design of geotechnical features and foundations can be downloaded from the following FHWA web site:

http://www.fhwa.dot.gov/engineering/geotech/pubs/nhi11032/nhi11032.pdf

3.3.2 Final Foundation Report

Update and revise the Draft Foundation Report; incorporate review comments received from DPW and key stakeholders on the Draft Foundation Report; and then generate and deliver the sealed Final Foundation Report to the Department.

3.3.3 Draft Foundation Data Sheets

Prepare drawings that include presentation of plan and profile to scale of subsurface data, insitu testing results, special details, and classification of subsurface materials. Foundation report shall include bearing capacity calculations of footings and piles including settlement calculations/estimate. Indicate pile driving parameters and formulas used in calculating pile capacities as well as pile embedment requirements.

3.4 Hydraulic Analysis and Report

The preferred design criteria for determining the bridge waterway opening includes:

- Provide two feet of freeboard between the river water surface elevation resulting from the peak 50-year flood discharge and the lowest elevation on the underside of the superstructure.
- For coastal bridges, provide two feet of freeboard between the storm surge plus wave height elevation that results from a 50-year recurrence interval storm. For Guam, that elevation may be taken to be 13.4 feet islandwide, based on an MSL Storm Surge Elevation of 7.4 feet and Breaking Wave Height of 4.0 foot published

by the U. S. Army Corps of Engineers in the 'Agana Bay Typhoon and Storm-Surge Protection Study, January 1984.

- Pass the 100-year flood without overtopping the bridge or adjacent roadway.

It is not uncommon for coastal bridges on Guam that the profile grade of the adjacent roadway will not accommodate the preferred freeboard values. In that event, the bridge should be designed to accommodate the adjacent roadway profile grade with a superstructure that is as thin as can be reasonably achieved for the required span arrangement, and the structure shall be designed to resist the lateral and uplift forces that result from inundation of the superstructure by river flooding and ocean surges including waves.

For bridge replacement projects, the proposed bridge shall not result in an increase to the 100-year base flood elevation that is computed or published for the existing bridge.

For a new roadway, the proposed bridge shall not result in an increase to the 100-year base flood elevation of more than 1.0 foot over the natural condition 100-year flood that is computed or published for the site without the proposed roadway and bridge.

Modification of existing stream channels below the Ordinary High Water elevation as determined in accordance with U. S. Army Corps of Engineers procedures is strongly discouraged.

3.4.1 Modeling

Prepare a hydraulic model which shall include the following: natural (existing) conditions; the proposed bridge or other project structure; bridge backwater for required peak flows; overtopping flood (if occurring prior to 100-year). Determine and estimate design flows for the 50-year (design) and 100-year (base) flood events.

3.4.2 Scour Analysis/Countermeasures

Use Federal Highway Administration ("FHWA") methodology in the computation of scour-depth; and compare historical survey data (if available) to determine changes in channel geometry and streambed elevation, and shall use channel thalweg elevation for scour depth measurement base line when dealing with channels that have tendencies for lateral migration.

3.4.3 Draft Hydraulics Report

Prepare a Draft Hydraulics Report that includes the following:

- Description of the stream under the bridge
- Site constraints
- Notation of visible problem areas including lateral-channel stability and signs of stream migration that could affect stability for piers; bents or abutments
- Notation of degradation (head-cutting) or aggradation (deposits) in the channel
- Manning's "n" value for the main channel and overbank areas, documented with color photographs
- Note size of existing riprap at abutments and piers and note any riprap failure.
- Determine bed material size by visual inspection for values of variables in scour prediction equations

- Note and record evidence of scour; note and record pier alignment (skewed or normal to flow)
- Note and record hydraulic controls from channel constrictions, and other features
- Note and record apparent or observed high-water marks
- Note and record evidence of debris; record conversations with local residents, and DPW personnel

Additionally, provide the following: the bank full width, water elevation, flow area, backwater and discharge for the design flood and 100-year base flood. Any environmental constraints (e.g., threatened or endangered species) that may limit the use of scour protection (e.g., riprap) should be noted.

3.4.4 Final Hydraulics Report

Update and revise the Draft Hydraulics Report; incorporate review comments received from DPW and key stakeholders and deliver the sealed Final Hydraulics Report to the Department.

4.0 REHABILITATION AND REPAIR

The technology of bridge rehabilitation and repair is constantly changing. In addition, many of the defects encountered vary from bridge to bridge requiring individual unique solutions. Consequently, this section merely presents an overview of bridge rehabilitation and some of the more common types of repairs. The repairs that are discussed are all proven to be reasonably successful and are approved by FHWA for use on federally funded projects.

4.1 Design Considerations

For individual members, it will be necessary to determine whether the best option is to repair or replace. In making this decision, cost shall be considered along with factors such as traffic maintenance , convenience to the public, longevity of the structure, whether the rehabilitation is long term or short term, and the practicality of either option.

Due to the variation in the types of problems encountered, the designer shall perform an in depth inspection of the structure to identify the defects that exist, and develop a solution which is unique to the problems found. This field inspection should include color photographs and sketches showing pertinent details and field verified dimensions.

It is imperative that an in depth, hands on, inspection of bridges be made, by the designer preparing the repair or rehab plans, to determine the extent of structural steel and concrete repairs. This inspection shall be made concurrent with plan development. Large quantity and cost overruns can result when this inspection is not adequately performed, resulting in substantial delays to completion of the project.

Pertinent dimensions that can be physically seen shall be field verified or field measured by the designer and incorporated into the plans. It is not permissible to take dimensions directly from old plans without checking them in the field because deviations from plans are common. Every attempt shall be made to prepare plans that reflect the actual conditions in the field. However, it is recognized that uncertainties may exist. Consequently, a note calling for existing structure verification should be included in the plans with the understanding that the designer is still responsible for making a conscientious effort to provide accurate information based on field observations.

4.2 Strength Analysis

When analyzing existing superstructures, substructures and foundations for strength, the live load is to be the HS20-44 truck (or lane load) or the alternate military loads for specified routes.

In analyzing the strength of existing superstructures, substructures and foundations for bridges that are to receive a new deck, a future wearing surface of 30 psf shall be included in the dead load.

New elements such as a superstructure replacement shall be designed for current design loads even though the foundations that are retained may have been designed for a lesser live load. The designer should minimize the dead load of a new superstructure to maximize the live load capacity of existing foundations that are retained. The new superstructure shall not exceed the weight of the existing superstructure by more than 5 percent.

4.3 Seismic Retrofitting and Storm Surge Forces

Major rehabilitation projects such as deck replacement and superstructure replacement shall include modifications to retrofit the structure to provide resistance to lateral and uplift forces that result from seismic events. Details to provide seismic restrain may include a combination of abutment shear keys, converting abutment bearings to semi-integral joints, or providing tie-downs to connect the superstructure to the substructure. Refer to Section 2 of this document for design guidance.

Major rehabilitation projects for bridges subject to inundation from coastal storms shall also include modifications to provide resistance to lateral and uplift forces that result from coastal storm events. The same retrofit details that provide seismic resistance often provide satisfactory resistance to coastal storm events. Refer to Section 3 of this document for design guidance.

4.4 Damage or Section Loss

Main load carrying members such as decks, stringers, and pier caps that have experienced damage or section loss that adversely impacts their load carrying capacity and/or remaining useful life should not be retained in the rehabilitated structure. Where concrete spalling and other damage does not adversely impact load carrying capacity of a bridge member and where patching can effectively extend the remaining useful life (such as for abutment walls, pier columns, etc.), the existing member can be retained in the rehabilitated structure. It is the designer's responsibility to evaluate the repair areas and determine the most suitable repair method.

To serve as a guide to the designer, the following criteria have been established to help in the patching selection evaluation.

Formed or trowled concrete patches should be used where the repair depth is 3 inches or greater and the surface can be readily formed and concrete placed. This type of patch is the most durable due to its depth and the utilization of reinforcing bars to tie it together. This type of repair is typically paid on a square yard basis. Where extensive curb repair is encountered, the patching should be paid for on a lineal foot basis.

Pneumatically Placed Mortar, sometimes referred to by the proprietary name Shotcrete, should generally be used where the repair surface cannot be readily formed and concrete placed, where the depth of repair is between 1 and 6 inches, and where at least 150 square feet of repair area is involved.

The detail plans shall show and detail the locations of the areas that require patching repairs. Additionally, provide a plan note requiring the surfaces to be patched and the exposed reinforcing steel to be abrasively cleaned within 24 hours of application of patching material (or erection of forms if the forms would render the area inaccessible to blasting).

Trowelable mortar should generally be specified when the repair depth is less than 1½ inches deep, and the repair area is less than 150 square feet. Trowelable mortar should also be specified in lieu of pneumatically placed mortar for the case where the depth of patch is equal to or less than 3 inches and the quantity is less than 150 square feet. Three inches is the maximum depth of patch that should be attempted with this type of mortar.

Cracks can be repaired by epoxy injection. The location of the cracks shall be shown in the plans and marked in the field.

The designer shall outline the areas to be repaired on the structure and also show where these areas are on details in the plans. Include appropriate specifications in SCR 552.

4.5 Bridge Deck Repair

Bridge decks that exhibit significant spalling and delamination (greater than 15% of deck area) typically have chloride levels such that repairs will not last long. Repair of decks is not recommended for major rehabilitation of a bridge.

If a repair is considered for an existing structure, chloride testing shall be done on the existing concrete bridge deck and other major elements, using AASHTO T260, to fully assess the structural condition and life expectancy of the element.

The request for testing should be submitted to DPW, early in the design process to allow adequate time for collection and testing of the samples.

In order for rehabilitation to be considered as a viable alternative the chloride content of the existing concrete surface of the major structural bridge elements must be low (less than or equal to 0.015% by mass of samples). If the chloride content of the existing concrete surface is high (0.015% or greater by mass of samples), then consideration should be made of the types of elements that have tested high. It still may be possible to remove and replace the contaminated concrete, but must be evaluated economically against replacing the bridge.

Where local laboratories are unable to perform Chloride Testing, the concrete samples shall be sent to an accredited laboratory in the United States for testing and analysis. A copy of the certified laboratory test report shall be filed with the PMT and DPW.

4.6 Bridge Deck Replacement

Provide appropriate plan notes to call for deck removal in order to prevent damage to existing stringers that are to remain in the bridge.

Superelevated deck sections (existing and new) may need temporary modifications to the slope of the deck and/or shoulder in order to accommodate the traffic from the phased construction. The designer is to make this determination during the Structure Type Study and add additional details and/or notes as necessary. Structural members may require additional structural analysis to insure their adequacy and that no damage to the member will occur.

On all deck replacement projects, the elevations of the bottom of the beam shall be field determined so that when the deck is built to the new plan profile grade, it will be possible to obtain the required minimum deck thickness. Elevations shall be taken at the beam seats and in the interior portions of the spans. This is a design consideration and is not something that should be left for the contractor to deal with after a contract has been awarded.

If possible, a 2 inch haunch depth should be provided over the stringers unless this haunch would cause undue problems with the profile grade off the bridge.

It is sometimes necessary to raise the profile grade of a structure. One way to accomplish this change when replacing the deck is by using deep haunches. The maximum recommended haunch depth is 12 inches. Provide reinforcing steel in any haunch greater than 5 inches. A deep haunch (5 inches or more) shall be made with the horizontal haunch width limited to 9 inches on either side of the stringer flange.

A closure pour may not be necessary for replacement of a deck on existing stringers even when using stage construction since differential deflections will be resisted by the existing cross frames.

If a deck replacement project also includes an integral or semi-integral retrofit at the abutments, a closure pour may be required for longer span bridges. New concrete abutment diaphragms without a closure pour at the stage line, will not allow the unloaded existing beams to freely deflect during the deck replacement pour.

4.7 Expansion Joint Retrofit

While it is desirable to seal the expansion joint of bridges, it is not desirable to demolish a functional expansion joint and possibly a backwall simply for the purpose of installing a seal. As long as a severe corrosion problem does not exist, additional coating will preserve the components exposed to the expansion joint discharge until the deck is replaced. However, it shall in fact be established that a severe problem does not exist if coating is the chosen course of action.

On more extensive projects, where the deck is being replaced, consider using semi-integral design. This type of design can be used for bridges whose foundations are stable and fixed (for example on two rows of piles). Full integral details shall be used when the foundation consists of a single row of piles. Additional considerations are that the geometry and layout of the approach slab, wingwalls, curbs, sidewalks, utilities and transition parapets shall be compatible with (not restrain) the anticipated longitudinal movement. For example approach slabs would have to move independently of turned back wings since the superstructure and approach slab move together. If the approach slab were connected to turned back wings in any manner, then movement of the entire superstructure would be restricted.

4.8 Bridge Drainage

Much damage has occurred on bridges as a result of poorly designed drainage. Proper drainage is extremely important to the longevity of the structure. All dysfunctional drainage systems should be retrofitted. Consequently, the designer shall give adequate attention to the development and presentation of correct details for this important function.

If it is found that existing scuppers are not necessary, and the deck is not being replaced, they should be plugged. If the scuppers are plugged, the additional drainage directed off the bridge shall be collected. If the deck is being replaced, the scuppers should be removed.

Existing functional scuppers may need to be extended so that they are 8 inches [200 mm] below the bottom flange. Check to see if the bottoms are rusted through before preparing the scupper extension detail.

4.9 Bridge Widening

4.9.1 Superstructure

The widened section should be designed so that superstructure deflections for the new and old portions are similar. No single rule is applicable for the necessity for closure pours on a widening project. The flexibility of each member, the overall theoretical deflection and use of integral or semi-integral abutments will cause each project to be unique. The purpose of the closure pour is to accommodate the differences in deflection that can occur between the new and the old during phased construction.

For widening projects where the existing deck is removed and a new or wider, deck is being placed, with no superstructure members added, no closure pour is necessary.

For widening projects (2 beams or more) where either the existing deck is to remain or the phase line of a new deck will be between the existing and new superstructure, a closure pour should be provided. Cross frames in the bay between the new and existing superstructure should not be welded until after the phase 1 and 2 new deck portions have been placed. After the cross frames have been welded, the closure section, phase 3, can be completed. Reinforcing steel splices should occur within the closure section. The width of the closure section should be at least 30 inches.

For widening projects (2 beams or more) where the deck's phase line is not between the new and existing superstructure members, a closure pour will still be required. Existing cross frames under the closure pour location need to be released before the phase 1 deck removal begins. Cross frames between new and existing members should be installed before the phase 2 pour. Re-install the released crossframes after the phase 2 pour but before the phase 3 pour. Reinforcing steel splices should occur within the closure section. The width of the closure section should be at least 30 inches.

For widening projects (2 beams or more) where a new deck is being constructed but the phase line is between existing superstructure members and at least 3 bays away from new member locations, a closure pour is still required. The procedure for crossframe release should be the same as defined in the paragraph above. The closure pour may be eliminated for this condition if the designer can show that the outside existing member, now being attached to the new member, is not restrained from returning to its original unloaded position by the new cross frames.

Closure pours may be eliminated if the differential deflection is expected to be less than ¼ inch, regardless of superstructure type.

In special cases, the minimum closure section width may be reduced by the use of mechanical connectors. The designer should not blindly apply this exception since the use of lap splices is preferred and recommended.

Falsework for the new slab should be independent and not be tied to the original superstructure. This would not apply to falsework for the closure section. The release of falsework for reinforced concrete slab superstructures may need to be coordinated (i.e. specified in the plans) between phases in certain situations.

Closure pours on bridge structures with integral or semi-integral abutments shall include the abutment's diaphragm concrete. Any concrete pier diaphragm shall also be included in the closure pour.

4.9.2 Foundations for Widened Structures

Differential foundation settlements shall be considered. For example, if it is required to widen a bridge adjacent to an existing spread footing, it is possible that the existing foundation has settled as much as it is going to. However, if the widened portion is placed on a new spread footing, then that portion will settle with respect to the original and distress to the structure will result. Consequently, the new portion should be placed on piling or drilled shafts in an attempt to limit differential settlement.

4.10 Scour Considerations

Substructure foundations need to be investigated for scour. The investigation consists of determining what the substructures are founded on; how deep the foundation is; and a decision on whether potential scour will endanger the substructure's integrity. Scour due to ocean storm surges, local scour and stream meander need to be considered.

4.11 Railing

Railing not meeting current standards will require upgrading when that structure is included in a major rehabilitation project.

The following sections are suggested methods for upgrading non-crash tested railing.

Facing – This method works when the existing parapet is in relatively good condition. The existing parapet and safety curb can be partially removed and a facing section placed on top. Dowels should be at least 6 inches deep and should be spaced at no more than 15 inches c/c. Grout should be epoxy grout. Details showing removal of existing concrete, dimensions for placement of new concrete, treatment of the parapet at the expansion joint, parapet transition details, typical sections, joint spacing, reinforcing steel, limits for purpose of measurement and payment, and what pay item the work is to be included with are also required.

Removal Flush with the Top of the Deck – If the outside of the existing parapet is in poor condition, the parapet and curb can be sawn off and a new parapet installed. Dowels should be at least 6 inches deep and should be spaced at no more than 15 inches c/c. The basis for this depth and spacing is research report FHWA-CA-TL-79-16 prepared by CalTrans in June of 1979 where they performed crash testing of various railing sections with shallow rebar anchorage. Grout should be epoxy per CMS 705.20. It will be necessary to call for epoxy grout as other materials are also covered in these specifications.

5.0 FINAL DESIGN

5.1 Pre-Final Design

The preparation of the pre-final bridge design documents follows the approval of the Final TS&L package. The design document produced should be detailed as required to produce a complete set of construction drawings, bid book with special contract requirements, construction cost estimate, and a construction schedule. Notify the Department immediately if it is determined that the estimated construction cost or the estimated construction time will exceed the programmed cost and timeframe so that appropriate revision can be made to either the scope of the improvements, or to the programmed costs or schedule. Submit the pre-final bridge plans with the 95% roadway submittal.

5.1.1 Bridge General Notes

This section contains various typical general notes. The designer needs to assure that the typical notes are complete and apply to the specific project. These notes may need to be revised or specific notes must be written to conform to the actual conditions that exist on each individual project.

The designer shall include the following note specifying the design specifications used for design of the structure. If the note is not correct (i.e. in the case of rehabilitation of an existing structure), then the note should be revised with the correct criterion that describes the design specifications for the structure.

> DESIGN SPECIFICATIONS: This structure conforms to the "LRFD Bridge Design Specifications" adopted by the American Association of State Highway and Transportation Officials, XXXX*, including the XXXX (if any)* Interim Specifications
>
> *Designer should fill in current edition and latest interims.

Design of bridges with non-redundant components is discouraged. For bridges with non-redundant components, the following note shall be included:

> REDUNDANCY: The following item(s) were considered non-redundant for design and include a load modifier equal to 1.05 in accordance with the AASHTO LRFD Bridge Design Specifications, Article 1.3.4:
>
> - Include a list of all items considered non-redundant

For bridges with non-redundant foundation components, the following notes shall be included:

> REDUNDANCY: The piles supporting the following substructure(s) were considered non-redundant for design and include a modified resistance factor equal to (1) in accordance with the AASHTO LRFD Bridge Design Specifications, Article 10.5.5.2.3: Include a list of all items considered non-redundant.

REDUNDANCY: The drilled shafts supporting the following substructure(s) were considered non-redundant for design and include a modified resistance factor equal to (1) in accordance with the AASHTO LRFD Bridge Design Specifications, Article 10.5.5.2.4:

- Include a list of all substructures with pile foundations or drilled shafts considered non-redundant for design in accordance with AASHTO LRFD 10.5.5.2.3 & 10.5.5.2.4.
- (1) Provide the modified resistance factor value. This should be equal to 80% of the resistance factor used for design on redundant pile foundations.

For all bridges the following note shall be included:

OPERATIONAL IMPORTANCE: A load modifier of XXXX has been assumed for the design of this structure in accordance with the AASHTO LRFD Bridge
Design Specifications, Article 1.3.5.

For bridges designed for highway loads, the design loading shall be provided in the following note:

DESIGN LOADING: HL-93
Future Wearing Surface (FWS) of 0.030 kips/ft^2

For bikeway/pedestrian bridges that will not accommodate vehicular traffic the design loading shall be provided in the following note:

DESIGN LOADING: 0.090 kips/ft^2

For bikeway/pedestrian bridges subject to emergency vehicular traffic the design loading shall be:

DESIGN LOADING: 0.090 kips/ft^2 and H15-44 vehicle

Provide a note listing general design data:

DESIGN DATA:
Concrete Class C – Compressive strength 4.5 ksi (superstructure)
Concrete Class A – Compressive strength 4.0 ksi (substructure)
Concrete Class A – Compressive strength 4.0 ksi (drilled shaft)
Reinforcing steel - minimum yield strength 60 ksi
    Concrete for prestressed beams:
    Compressive Strength (final) – 6.0 ksi
Compressive Strength (release) – 4.5 ksi
Prestressing Strand:
    Area = 0.153 in2
    Ultimate Strength = 270 ksi
    Tensioning Stress = 202.5 ksi (Low relaxation strands)
Structural Steel - ASTM A709 Grade (1) - yield strength (1) ksi

Note to Designer: Modify note as necessary. Delete references that are not applicable to project. (1) Grade 50 - yield strength 50 ksi, or Grade 50W - yield strength 50 ksi, or Grade HPS70W - yield strength 70 ksi. If more than one grade of steel is selected, the description shall clearly indicate where the different grades are used in the structure.

The following sample notes will serve as a guide in composing the note(s) for the removal of the existing structure. Modify the notes as required to fit the conditions. Use the following note if it is the desire of the Department to salvage any portion of the bridge (this is not common).

REMOVAL OF EXISTING STRUCTURE: Carefully dismantle the XXXX and store along the right-of-way for disposal by the State's forces.

Note to Designer: Describe the degree of care to be exercised in the removal in sufficient detail to allow accurate bidding. If this option is used, the pay item shall be "as per plan".

Use the following note when portions of the structure will be used to maintain traffic during phased construction and the stream below must be protected from falling debris:

REMOVAL OF EXISTING STRUCTURE: When no longer needed to maintain traffic, portions of the existing structure shall be removed in phases as detailed on the plans. Removal shall be accomplished by sawcutting and lifting segments from the bridge or by use of scaffolding erected beneath the bridge to prevent debris that is removed from the bridge from falling into and fouling the stream beneath the bridge.

Note to Designer: Add the following for bridge rehabilitation projects: Existing concrete shall be sawcut 1" deep at margins where proposed work will join to existing concrete structures at visible locations.

Add the following when removal of structure as specified in FP-03 Section 203 will not fill the specific requirements of the project: Remove abutments to Elev. XXX. Remove piers to Elev. XXX.

Us the following note when traffic must be maintained on a roadway adjacent to deep excavation required to construct the bridge:

The contractor shall install temporary sheet piling shoring as needed to accommodate excavation and removal of existing structure elements for construction of the proposed bridge adjacent to the traveled way. All temporary shoring and falsework shall meet the requirements of AASHTO Guide Design Specifications for Bridge Temporary Works and Construction Handbook for Bridge Temporary Works. Sheet piling shall be removed when no longer needed. Include the cost for material and labor necessary to install, maintain, and remove the temporary sheeting and bracing in the lump sum price bid for Shoring and Bracing.

For all substructure units where embankment construction is involved, provide appropriate embankment construction notes in the Structure General Notes. The following construction method should minimize the effect of lateral forces acting on substructure units and their piles. For structures with abutments on piles placed in new embankments use the following note:

> PILE DRIVING CONSTRAINTS: Prior to driving piles, construct the spill through slopes and the bridge approach embankment behind the abutments up to the level of the subgrade elevation for a minimum distance of (1) behind each abutment. Do not begin the excavation for the abutment footings and the installation of the abutment piles until after the above required embankment has been constructed.
>
> *Note to Designer: (1) Distance is generally 200 feet. Optionally, this distance may be defined by station-to-station dimensions.*

For structures with abutments and piers on piles placed in new embankments use the following note:

> PILE DRIVING CONSTRAINTS: Prior to driving piles, construct the spill through slopes and the bridge approach embankment behind the abutments up to the level of the subgrade elevation for a minimum distance of (1) behind each abutment. Do not begin the excavation for the abutment footings and the installation of the abutment and pier piles, for pier(s): (2), until after the above required embankment has been constructed.
>
> *Note to Designer: (1) Generally 200 feet. Optionally, this distance may be defined by station-to-station dimensions. (2) Identify specific piers.*

For structures with wall type abutments on piles placed in new embankment use the following note:

> PILE DRIVING CONSTRAINTS: Prior to driving piles at the abutments, construct the bridge approach embankment behind the abutments up at a 1:1 slope from the top of the heel of the footing (1) to the subgrade elevation and for a minimum distance of 250 feet behind the abutments. Do not begin the installation of the abutment piles until after the above required embankment has been constructed. After the footing and the breastwall have been constructed, construct the embankment immediately behind the abutments up to the beam seat elevation and on a 1:1 slope up to the subgrade elevation prior to setting the beams on the abutments.
>
> *Note to designer: (1) In some cases the bottom of the heel may be used.*

For foundations on spread footings in new embankments, the following construction method helps to eliminate any lateral forces on the foundation due to the construction of the embankment and/or settlement of the subgrade under the embankment. For stub abutments on spread footings being constructed in new embankments provide the following note:

CONSTRUCTION CONSTRAINTS: Prior to constructing the spread footing foundations, construct the bridge approach embankments behind the abutment up at a 1:1 slope from the bottom of the heel of the footing to the subgrade elevation and for a minimum distance of 250 feet behind the abutments. After the abutment footing and breastwall are completed and prior to setting superstructure members, construct the embankment immediately behind the abutment up to the beam seat elevation and on a 1:1 slope up to the subgrade elevation, with Type B granular material conforming to 703.16.C.

Note to Designer: Modify the note, as appropriate, for piers constructed on a spread footing foundation.

The following note generally will apply where piles are to be driven to bedrock:

PILES TO BEDROCK: Drive piles to refusal on bedrock. The Department will consider refusal to be obtained by penetrating weak bedrock for several inches to a minimum resistance of 20 blows per inch or by contacting strong bedrock and the pile receiving at least 20 blows. Select the hammer size to achieve the required depth to bedrock and refusal. Instead of driving to refusal, the Contractor may perform dynamic load testing to establish driving criteria for each pile type and capacity. Establish the driving criteria to achieve an Ultimate Bearing Value that is 1.5 times the total factored load given below for the piles. Payment for dynamic load testing performed at the Contractor's option is included in the unit price pay item for piles driven. The total factored load is (1) kips per pile for the (2) abutment piles. The total factored load is (1) kips per pile for the (2) pier piles.

Note to Designer: (1) Specify the total factored load. (2) Specify the location of piles for each total factored load.

The following note, modified to fit the conditions, will apply where piles are located within a waterway and the scour depth is significant.

PILES TO BEDROCK: Drive piles to refusal on bedrock. The Department will consider refusal to be obtained by penetrating weak bedrock for several inches to a minimum resistance of 20 blows per inch or by contacting strong bedrock and the pile receiving at least 20 blows. Select the hammer size to achieve the required depth to bedrock and refusal. Instead of driving to refusal, the Contractor may perform dynamic load testing according to C&MS 523 to establish driving criteria for each pile type and capacity. Establish the driving criteria to achieve an Ultimate Bearing Value that is 1.5 times the total factored load given below for the piles. Payment for dynamic load testing performed at the Contractor's option is included in the unit price pay item for piles driven.

The total factored load is (1) kips per pile for the (2) abutment piles. The abutment piles were designed to accommodate (3) ft. of scour. The total factored load is (1) kips per pile for the (2) pier piles. The pier piles were designed to accommodate (3) ft. of scour.

> Note to designer: (1) Specify the total factored load. (2) Specify the location of piles for each total factored load. (3) Specify the depth of anticipated scour.

The following note, modified to fit the conditions, will apply where downdrag loads on the piles are anticipated.

> PILES TO BEDROCK: Drive piles to refusal on bedrock. The Department will consider refusal to be obtained by penetrating weak bedrock for several inches to a minimum resistance of 20 blows per inch or by contacting strong bedrock and the pile receiving at least 20 blows. Select the hammer size to achieve the required depth to bedrock and refusal. Instead of driving to refusal, the Contractor may perform dynamic load testing to establish driving criteria for each pile type and capacity. Payment for dynamic load testing performed at the Contractor's option is included in the unit price pay item for piles driven.
>
> The total factored load is (1) kips per pile for the (2) abutment piles. The abutment piles include an additional (3) kips of factored load per pile to account for possible downdrag loading. The total factored load is (1) kips per pile for the (2) pier piles. If performing dynamic load testing to establish driving criteria, the Ultimate Bearing Value is (4) kips per pile for the abutment piles and (4) kips per pile for the pier piles.
>
> *Note to Designer: (1) Specify the total factored load. (2) Specify the location of piles for each total factored load. (3) Specify the anticipated factored downdrag loading. (4) Specify the Ultimate Bearing Value for dynamic load testing, including downdrag.*

For friction piles that are not driven to bedrock, the following notes, modified to fit the specific conditions for the foundation required, will apply. Provide the actual calculated Ultimate Bearing Value as shown below:

> PILE DESIGN LOADS (ULTIMATE BEARING VALUE): The Ultimate Bearing Value is (1) kips per pile for the (2) abutment piles. The Ultimate Bearing Value is (1) kips per pile for the (2) pier piles.
>
> *Note to Designer: (1) Specify the Ultimate Bearing Value. (2) Specify the location of piles for each Ultimate Bearing Value.*

The following note, modified to fit the conditions, will apply where friction piles are located within a waterway and scour is anticipated.

> PILE DESIGN LOADS (ULTIMATE BEARING VALUE): The Ultimate Bearing Value is (1) kips per pile for the (2) abutment piles. The Ultimate Bearing Value is (1) kips per pile for the pier piles. The pier piles include an additional (3) kips per pile of Ultimate Bearing Value due to the possibility of losing (4) ft. of frictional resistance due to scour.
>
> *Note to Designer: (1) Specify the Ultimate Bearing Value. (2) Specify the location of piles for each Ultimate Bearing Value. (3) Specify the additional amount of Ultimate Bearing Value. (4) Specify the scour depth.*

The following note, modified to fit the conditions, will apply where downdrag loads on the friction piles are anticipated.

> PILE DESIGN LOADS (ULTIMATE BEARING VALUE): The Ultimate Bearing Value is (1) kips per pile for the (2) abutment piles. The Ultimate Bearing Value is (1) kips per pile for the (2) pier piles. The addition of (3) kips of Ultimate Bearing Value per abutment pile is due to possible downdrag loads caused by settlement/liquefaction and to account for side friction within the downdrag zone that must be overcome during pile driving.
>
> *Note to Designer: (1) Specify the Ultimate Bearing Value. (2) Specify the location of piles for each Ultimate Bearing Value. (3) Specify the additional amount of Ultimate Bearing Value. This amount includes the factored downdrag load and the unfactored side resistance from the soil in the downdrag zone.*

Provide the following note when Static Load Testing is required for friction piles. Modify the note as necessary to fit the specific condition.

> STATIC LOAD TEST: Perform dynamic testing on the first two production piles to determine the required blow count for the specified Ultimate Bearing Value. Perform the static load test on either pile. Do not over-drive the selected pile. Drive the third and fourth production piles to 75% and 85% of the determined blow count, respectively and perform dynamic testing on each. The test piles and the reduced capacity piles shall not be battered. After installation of the first four production piles, cease all driving operations on piling represented by the static load testing for a minimum of 7 days. After the waiting period, perform pile restrikes on the four piles (two restrike test items). The Engineer will review the results of the pile restrikes and establish the driving criteria for the remaining piling represented by the testing. Submit all test results to the Department.
>
> For subsequent static load tests, upon completion of a 10,000 ft increment of driven length, repeat the above procedure for the initial static load test. If necessary, the Engineer will revise the driving criteria for the remaining piling accordingly.
>
> When performing the restrike, if the pile has not reached the blow count determined for the plan specified Ultimate Bearing Value, continue driving the pile until this capacity is achieved.

Provide the following note when battered friction piles are specified.

> BATTERED PILES: The blow count for battered piles shall be the blow count determined for vertical piles of the same Ultimate Bearing Value divided by an efficiency factor (D). Compute the efficiency factor (D) as follows:
>
> $$D = \frac{1-UG}{\sqrt{1+G^2}}$$
>
> U = Coefficient of friction, which is estimated at 0.05 for double-acting air operated or diesel hammers; 0.1 for single-acting air operated or diesel hammers; and 0.2 for drop hammers.
> G = Rate of batter (1/3, 1/4, etc.)

The following note, modified to fit the specific conditions for the foundation required, will apply when uplift loads control the design of the pile. In this case, the piles are typically driven to a pile tip elevation and dynamic load testing of the pile is not performed.

> PILES DRIVEN TO TIP ELEVATION FOR UPLIFT: Drive the piles to the pile tip elevation shown on the plans. Do not perform dynamic load testing on piles driven to a tip elevation. Select the hammer size to achieve the required depth. Provide plain cylindrical casings with a minimum pile wall thickness of (1) inch for piles driven to a tip elevation.
>
> *Note to Designer: (1) Specify the minimum pile wall thickness for cast-in-place reinforced concrete piles. Determine the minimum pile wall thickness from a pile drivability analysis. Remove this sentence if the piles are precast concrete.*

When abutments or piers are supported by spread footings on soil, include the following note to require that reference monuments be constructed in each footing. The purpose of the reference monuments is to document the performance of the spread footings, both short and long term.

> ITEM 552, CLASS A CONCRETE, AS PER PLAN: In addition to the requirements of Item 552, install a reference monument at each end of each spread footing. The reference monument shall consist of a #8, or larger, epoxy coated rebar embedded at least 6 inch into the footing and extended vertically 4 to 6 inches above the top of the footing. Install a six inch diameter, schedule 40, plastic pipe around the reference monument. Center the pipe on the reference monument and place the pipe vertical with its top at the finished grade. The pipe shall have a removable, schedule 40, plastic cap. Permanently attach the bottom of the pipe to the top of the footing.
>
> Establish a benchmark to determine the elevations of the reference monuments at various monitoring periods throughout the length of the construction project. The benchmark shall be the same throughout the project and shall be independent of all structures.
>
> Record the elevation of each reference monument at each monitoring period shown in the table on the following page.

Project Number:*	Maximum Factored Bearing Pressure*	
Bridge Name:*		
Benchmark Location		
Footing Location:*		
Monitoring Period	Left Monument	Right Monument
After footing concrete is placed		
Before placement of superstructure members		
Before deck placement		
After deck placement		
Project completion		

Note to Designer: Modify items marked with an asterisk to describe the project number, bridge name, footing location, and maximum factored bearing pressure.

Provide the following note if the footing excavation is mainly bedrock and the footings are to be at an elevation no higher than plan elevation:

> FOOTINGS: Place footings in bedrock at the elevation shown.

Provide the following note where footings are to be founded in bedrock at an elevation no higher than plan elevation.

> FOOTINGS shall extend a minimum of 3 inches* into bedrock or to the elevation shown, whichever is lower.
>
> *Note to Designer: *Shall be greater than 3 inches if required by design considerations.*

Provide the following note where footings are to be founded in bedrock, and where the encountering of bedrock at an elevation considerably above plan elevation may make it desirable to raise the footing to an elevation not above the specified maximum in order to effect an appreciable saving:

> FOOTINGS shall extend a minimum of 3 inches* into bedrock. If necessary due to poor bedrock material, the footings should be lowered. If the low point of the bedrock surface occurs 2 feet or more above plan elevation, the final footing elevations may be raised, upon approval by the Engineer, but to an elevation not higher than ** feet. Stepping of individual footings will not be permitted unless shown on the plans.
>
> Note to Designer: *Shall be greater than 3 inches if required by design considerations. **The maximum elevation allowed should assure that minimum soil cover over the footing is obtained; clearance from the superstructure to the finished ground elevation meets standards; quality of bedrock material at that elevation is

adequate; and minimum embedment into the bedrock material will not be adversely affected.

Use the following drilled shaft notes when applicable for the specific project. Revise the note for the project conditions and the different drilled shaft designs, if any, on the project.

DRILLED SHAFTS: The maximum factored load to be supported by each drilled shaft is * kips at the abutments and * kips at the piers. This load is resisted by side resistance within a portion of the bedrock socket and also by tip resistance. The factored resistance developed by side resistance is * kips, assumed to act along the bottom * feet of the bedrock socket for the abutments and * feet of the bedrock socket for the piers. The factored resistance provided by the drilled shaft tip is * kips.

*Note to Designer: *Complete the loads and dimensions in this note. Abutment and Pier sections of the note should be removed or revised as required.*

Include a note to require chamfering of exposed concrete corners:

CONCRETE CHAMFER: Construct a ¾" chamfer for all exposed non-reentrant concrete corners.

Notes concerning maintenance of traffic often are required for bridge work, especially in phased construction projects. The designer is responsible for any bridge maintenance of traffic notes being coordinated with the project's overall maintenance of traffic plans. Any phased construction lane widths, temporary or construction vertical and horizontal clearances, or construction access requirements must match requirements in the project's maintenance of traffic plans. Prepare a general note as required to communicate maintenance of traffic requirements to the bridge contractor.

Include the following note in the Structural General Notes when a concrete parapet or railing is used.

CONCRETE PARAPETS: As soon as a concrete saw can be operated without damaging the freshly placed concrete, sawcut 1-1/4 inches deep control joints into the perimeter of the concrete parapet starting and ending at the elevation of the concrete deck. Place the sawcuts at a minimum of 6 feet and a maximum of 10 feet centers. Use an edge guide, fence, or jig to ensure that the cut joint is straight, true, and aligned on all faces of the parapet. The joint width shall be the width of the saw blade, a nominal width of 1/4 inch. Leave the bottom ½ inch of the inside and outside face unsealed to allow water to escape.

Add the following note to ensure proper seating of prestressed concrete box beams for skewed bridges.

> BEARING PAD SHIMS: Place 1/8 inch thick preformed bearing pad shims, plan area X inches by X inches, under the elastomeric bearing pads where required for proper bearing. Furnish two shims per beam. The Department will measure this item by the total number supplied. The Department will pay for accepted quantities at the contract price for Preformed Bearing Pads. Any unused shims will become the property of the Department.
>
> *Note to Designer: The plan area of the shim pad shall be the same as the elastomeric bearing.*

5.1.2 Bridge Detail Notes

This section contains various typical structural detail notes that should be placed on the appropriate detail sheets. The designer needs to assure that the typical notes are complete and apply to the specific project. These notes may need to be revised or specific notes must be written to conform to the actual conditions that exist on each individual project.

A neoprene sheet is required for waterproofing of the backside of the joint between the integral backwall and the bridge seat. Include the following note, which contains criteria for the installation of this seal, for all integral and semi-integral abutments. Plan details will be required to show location and dimensional position for installation.

> INTEGRAL ABUTMENT EXPANSION JOINT SEAL: Install a 3 foot wide neoprene sheet at locations shown in the plans. Secure the neoprene sheeting to the concrete with 1-1/4" x #10 gage (length x shank diameter) galvanized button head spikes through a 1 inch outside diameter, #10 gage galvanized washer. Maximum fastener spacing is 9 inches. Use of other similar galvanized devices, which will not damage either the neoprene or the concrete, will be subject to the approval of the Engineer.
>
> No separate payment will be made for materials and labor to install the neoprene sheets. Include the costs for material and labor o install the neoprene sheets with the unit price bid for structural concrete.
>
> Center the neoprene strips on all joints. For horizontal joints, secure the horizontal neoprene strip by using a single line of fasteners, starting at 6 inches, +/-, from the top of the neoprene strip. For the vertical joints secure the vertical neoprene strip by using a single vertical line of fasteners, starting at 6 inches, +/, from the vertical edge of the neoprene strip nearest to the centerline of roadway. For vertical joints, install 2 additional fasteners at 6 inches, center to center, across the top of the neoprene strip on the same side of the vertical joint as the single vertical row of fasteners is located.
>
> The vertical neoprene strips shall completely overlap the horizontal strips. Lap lengths of the horizontal strips that are not vulcanized or adhesive bonded, shall be at least 1 foot in length, or 6 inches in length if the lap is vulcanized or adhesive bonded. No laps are acceptable in vertically installed neoprene strips.

The neoprene sheeting shall be 3/32 inches thick general purpose, heavy-duty neoprene sheet with nylon fabric reinforcement. The sheeting shall be "Fairprene Number NN-0003", by E. I. DuPont De Nemours and Company, Inc., "Wingprene" by the Goodyear Tire and Rubber Company, or an approved alternate. The neoprene sheeting shall conform to the following:

Description of Test	ASTM	Requirement
Thickness, inches	D751	0.094 ± 0.01
Breaking Strength, Grab, lbs, minimum	D751	700 x 700 (Long. x Trans.)
Adhesive Strip, 1" wide x 2" long, lbs, min.	D751	9
Burst Strength, psi, minimum	D751	1400
Heat Aging, 70 Hr, 212^{o}F, 180^{o} bend without cracking	D2136	No cracking of coating
Low temp. brittleness, 1 Hr, -40^{o}F, bend around 1/4" mandrel	D2136	No cracking of coating

Note to Designer: Change "integral" to "semi-integral" as appropriate.

Provide one of the following porous backfill notes on the appropriate detail sheets.

POROUS BACKFILL WITH FILTER FABRIC, 2 feet thick shall extend up to the plane of the subgrade, to 1 foot below the embankment surface, and laterally to the ends of the wingwalls.

Or, for use when weep holes are specified:

POROUS BACKFILL WITH FILTER FABRIC, 2 feet thick shall extend up to the plane of the subgrade, to 1 foot below the embankment surface, and laterally to the ends of the wingwalls. Place two cubic feet of bagged No. 3 aggregate at each weephole. Include bagged aggregate with porous backfill for payment.

For structures that contain bearing anchors, place one of the two following notes on an appropriate abutment or pier detail sheet near the "Bearing Anchor Plan". Where the Contractor is allowed the option of presetting bearing anchors (or formed holes), or of drilling bearing anchor holes, provide the first note. Where drilling of anchors into the bridge seat is required, provide the second note. (Formed holes are not practical for prestressed concrete box beam bridges.)

BRIDGE SEAT REINFORCING, SETTING ANCHORS: Accurately place reinforcing steel in the vicinity of the bridge seat to avoid interference with the drilling of bearing anchor holes or the pre-setting of bearing anchors.

BRIDGE SEAT REINFORCING, SETTING ANCHORS: Accurately place reinforcing steel in the vicinity of the bridge seat to avoid interference with the drilling of anchor bar holes.

Where bridge seats have been adjusted to compensate for the vertical deformation of elastomeric bearings, place the following note with the necessary modifications on the appropriate substructure detail sheet.

BRIDGE SEAT ELEVATIONS have been adjusted upward X inches at abutments and X inches at piers to compensate for the vertical deformation of the bearings.

For a structure with concrete backwalls, deck joints and concrete decks supported on beams or girders, show an optional backwall construction joint at the level of the approach slab seat and provide the following note either on the appropriate abutment detail sheet.

BACKWALL CONCRETE: Do not place backwall concrete above the construction joint at the approach slab seat until after the deck concrete in the span adjacent to the abutment has been placed.

For a steel beam bridge with concrete backwalls and sealed deck joints employing superstructure support or armor steel of considerable stiffness where there is a possibility of individual beams being lifted off of their bearings in a clamping operation, a note similar to the following shall be provided:

INSTALLATION OF SEAL: During installation of the support/armor for the super structure side of the expansion joint seal, observe the seating of beams on bearings to assure that positive bearing is maintained.

For prestressed concrete box beam bridges where the placement of the wingwall concrete above the bridge seat needs to occur after the beams have been erected to allow for the tolerances of the beam fit-up and for beam erection clearances, provide the following note:

ABUTMENT CONCRETE: Do not place the abutment concrete above the bridge seat construction joint until the prestressed concrete box beams have been erected.

Charpy V-Notch (CVN) material is a requirement to help assure fracture toughness of main material. Designers using this note should understand not only why CVN is specified but what a main member is. For example, crossframes of curved steel structures, because they are actual designed members carrying liveload forces, are also main members. Designers are reminded they must indicate specific pieces, members, shapes, etc. that are main members. Place the following note on a structural steel detail sheet for bridges having main load-carrying members that must meet minimum notch toughness requirements:

CVN: Where a shape or plate is designated (CVN), furnish material that meets the minimum notch toughness requirements as specified in the table on the following page:

ASTM Designation	Thickness and Connection Method	Value Min CVN
A709 Gr. 36	Up to 4 in (102 mm) mechanically fastened or welded	15 ft-lb @ 40 °F (20 J @ 4 °C)
A709 Gr. 50 (A572), A709 Gr. 50W (A588/ A588M)	Up to 4 in (102 mm) mechanically fastened	15 ft-lb @ 40 °F[1] (20 J @ 4 °C)
A709 Gr. 50 (A572), A709 Gr. 50W (A588)	Over 2 to 4 in (51 to 102 mm) welded	20 ft-lb @ 40 °F[1] (20 J @ 4 °C)
A709 Gr. 50 (A572), A709 Gr. 50W (A588)	Up to 2 in (51 mm) welded	15 ft-lb @ 40 °F[1] (20 J @ 4 °C)
A709 Gr. 70W	Up to 4 in (100 mm) mechanically fastened or welded	2515 ft-lb @ -10 40 °F[1] (34 20 J @ -23 °C)
[1] If the yield point of the material exceeds 65 ksi (448 MPa), the temperature of the CVN value for acceptability should be reduced by 15 °F (8.3 °C) for each increment, or part of increment, of 10 ksi (69 MPa) above 65 ksi (448 MPa).		

For all structural steel superstructures, place the following note on the structural detail sheet:

> HIGH STRENGTH BOLTS shall be X diameter A325 unless otherwise noted.
>
> *Note to Designer: 7/8" diameter bolts are most commonly used*

Where the load plate of an elastomeric bearing is to be connected to the structure by welding, provide the following note with the pertinent bearing details:

> ELASTOMERIC BEARING LOAD PLATE WELDING: Control welding so that the plate temperature at the elastomer bonded surface does not exceed 300° F as determined by use of pyrometric sticks or other temperature monitoring devices.

For precast prestressed AASHTO girder bridges, use the following note:

> INTERMEDIATE DIAPHRAGMS: Do not place the deck concrete until all intermediate diaphragms have been properly installed. If concrete diaphragms are used, complete the installation of the intermediate diaphragms at least 48 hours before deck placement begins. Concrete shall be Class C.

For integral and semi-integral bridges, hardened concrete end diaphragms restrain the movement and rotation of beam/girder ends that occur during deck placement. This restraint will increase stress in both the beam/girder and diaphragm. Factors that can contribute to detrimental stress increases include large structure skew and phased construction. When these factors exist, hardened diaphragms should be avoided during the deck placement. The following table provides guidelines for concrete diaphragm placement options.

Designers should consider the absence of restraint at the diaphragm location and when calculating the unbraced length of steel beam/girder flanges. If necessary, temporary bracing details should be included in the plans. Temporary end bracing should be oriented perpendicular to beam/girder webs.

The following notes may be needed depending on whether the bridge superstructure is steel or prestressed concrete; requires phased construction; or is skewed a specific amount.

> Use the following note for either steel superstructures skewed less than 30 degrees or AASHTO girder superstructures skewed less than 10 degrees without phased construction.
>
> ABUTMENT DIAPHRAGM CONCRETE: Place the diaphragm concrete encasing the structural member ends with the deck concrete or at least 48 hours before placement of the deck concrete. If placed separately, locate the horizontal construction joint between the diaphragm and deck concrete at the approach slab seat.

Use the following note for either steel superstructures skewed 30 degrees or more or AASHTO girder superstructures skewed 10 degrees or more without phased construction.

> ABUTMENT DIAPHRAGM CONCRETE: Place the diaphragm concrete encasing the structural member ends after the deck placement in the adjacent span is complete. Procedures that place the abutment diaphragm with the deck concrete may be approved by the Engineer if the placement submittal can assure that the deck concrete in the adjacent span will be placed before concrete in the diaphragm has reached its initial set.

Use the following note for either steel superstructures skewed less than 30 degrees or I-beam superstructures skewed less than 10 degrees with phased construction and closure pours required.

> ABUTMENT DIAPHRAGM CONCRETE, PHASED CONSTRUCTION: Place the diaphragm concrete encasing the structural member ends of an individual phase with the deck concrete or at least 48 hours before placement of the deck concrete. If placed separately, locate the horizontal construction joint between the diaphragm and deck concrete at the approach slab seat. Place closure pour concrete in the diaphragm and deck concurrently.
>
> *Note to Designer: For bridges with phased construction that do not require closure pours, omit the last sentence of the above note.*

Use the following note for either steel superstructures skewed 30 degrees or more or I-beam superstructures skewed 10 degrees or more with phased construction and closure pours required.

> ABUTMENT DIAPHRAGM CONCRETE, PHASED CONSTRUCTION: Place the diaphragm concrete encasing the structural member ends of an individual phase after the deck placement in the adjacent span is complete. Procedures that place the abutment diaphragm with the deck concrete may be approved by the Engineer if the placement submittal can assure that the deck concrete in the adjacent span will be

placed before concrete in the diaphragm has reached its initial set. Place closure pour concrete in the diaphragm and deck concurrently.

Note to Designer: For bridges with phased construction that do not require closure pours, omit the last sentence of the above note.

For all steel beam and girder bridges with a concrete deck, provide the following note that describes how the quantity of deck concrete was calculated.

DECK SLAB CONCRETE QUANTITY: The estimated quantity of deck slab concrete is based on the constant deck slab thickness, as shown, plus the quantity of concrete that forms each beam/girder haunch. The estimate assumes a constant haunch thickness of __inches and a constant haunch width outside the edge of each beam/girder flange of 9 inches. Deviate from this haunch thickness as necessary to place the deck surface at the finished grade. The allowable tolerance for the haunch width outside the edge of each beam/girder flange is ± 3 inches.

The haunch thickness was measured at the centerline of the beam/girder, from the surface of the deck to the bottom of the top flange minus the deck slab thickness. The area of all embedded steel plates has been deducted from the haunch quantity.

Note to Designer: The note above applies to new structures with beams/girders placed parallel to the profile grade line. A constant depth haunch may not be practical for new structures whose beams/girders are not placed parallel to the profile grade line. In these special cases, the note shall be modified to fit the exact conditions.

For prestressed concrete I-beam bridges with concrete deck, compute the concrete topping depth over the top of the beams as follows:

A = Design slab thickness

B = Anticipated total midspan camber due to prestressing force at time of release

C = Deflection at midspan due to the self weight of the beam

D = Deflection at midspan due to dead load non-composite loads

E = Deflection at midspan due to dead load of composite dead loads (except FWS)

F = Adjustment for vertical curve. Positive for crest vertical curves

G = Sacrificial haunch depth (2")

H = Total Topping Thickness at beam bearings = A + 1.8B - 1.85C - D - E - F + G.
(If F> 1.8 B - 1.85C - (D+E) then H = A + G)

I = Total Topping Thickness at mid-span = A + G.
If F > 1.8B - 1.85C - (D +E) then I = A - (1.8B - 1.85C) + D + E + F + G

Use the gross moment of inertia for the non-composite beam to calculate the camber and deflection values B, C, and D. For E, use the moment of inertia for the composite section.

Show a longitudinal superstructure cross section in the plans detailing the total Topping Thickness including the design slab thickness and the haunch thickness at the centerline of spans and bearings. Also provide screed elevation tables. Provide the following note in the plans:

> Calculated camber at the time of release is X inches.
> Calculated camber at the time of erection is X inches.
> Calculated long-term camber is X inches.
>
> *Note to Designer: The camber at the time of release is (B-C), the camber at the time of erection is (1.8B – 1.85C), and the long-term camber is (2.45B- 2.40C).*

Provide the following note for AASHTO girder bridges:

> DECK SLAB THICKNESS FOR CONCRETE QUANTITY: The Topping thicknesses shown from the top of the deck slab to the top of the top flange along the centerline of the I-beam are theoretical dimensions. The haunch depth is the Topping thickness minus the design slab thickness. The Department will pay for superstructure concrete based on the design slab thickness and the average of the theoretical haunch depths at mid-span and at each beam bearing even though deviation from the dimensions shown may be necessary to place the deck surface at the finished grade. Once all beams are set in their final position, the actual camber for each member will be the top of beam elevation at mid-span minus the average top of beam elevation at each bearing. The actual Topping thickness at mid-span will be the theoretical dimension plus or minus the difference between the actual and anticipated camber.
>
> Use the following note when the length of the I-beam, measured along the grade, differs from the length, measured horizontally, by more than 3/8" [10mm]:
>
> NOTE TO FABRICATOR: The dimensions measured along the length of the beam, marked with a *, do not contain an allowance for the effect of the longitudinal grade. Include the proper allowance for these dimensions in the shop drawings.
>
> *Note to Designer: Indicate the dimensions that require a grade adjustment with an asterisk or some other easily recognizable symbol and include that symbol in the note above.*

For prestressed concrete box beam bridges, the asphalt or concrete topping depth over the top of the beams shall be computed as follows:
A = Minimum topping thickness
B = Anticipated total midspan camber due to prestressing force at time of release
C = Deflection due to the self weight of the beam (including diaphragms)
D = Deflection due to dead load of the topping and other non-composite loads
E = Deflection due to composite dead loads not including FWS
F = Adjustment for vertical curve. Positive for crest vertical curves

G = Total Topping Thickness at beam bearings = A + 1.8B - 1.85C - D - E – F.
If F > 1.8B - 1.85C - (D + E) then G = A
H = Total Topping Thickness at mid-span = A
If F > 1.8B - 1.85C - (D + E) then H = A - (1.8B - 1.85C) + D + E + F

Use the gross moment of inertia for the non-composite beam to calculate the camber and deflection values B, C, and D. For E, use the moment of inertia for the composite section when designing a composite box beam otherwise use the non-composite section. Note that with the exception of when F > 1.8B - 1.85C - (D + E), the dead load deflection adjustment (D + E) is made by adjusting the beam seat elevations upward.

For non-composite prestressed concrete box beam bridges with an asphaltic concrete surface course provide a note similar to the following:

Calculated camber at the time of release is X inches.
Calculated camber at time of paving is X inches.
Long term camber is X inches.
Calculated deflection due to dead load applied after the beams are set (weight of surface course, railings, sidewalks, etc.) is X inches.
The vertical curve adjustment to the topping thickness at midspan is X inches upward.
The vertical curve adjustment to the topping thickness at each bearing is X inches upward/downward.

Note to Designer: Conclude the note above with (1), (2), or (3) below:
1) The thickness of the intermediate asphalt course shall be 1½ inches. No variation in thickness is required.
2) The thickness of the intermediate asphalt course shall vary from 1½ inches at each centerline of beam bearing to X inches at midspan.
3) The thickness of the intermediate asphalt course shall vary from X inches at each centerline of beam bearing to 1½ inches at midspan.

For non-composite designs, include in the bridge plans a diagram showing the thickness of the intermediate variable course and the surface friction course at each centerline of bearing and at midspan.

For composite prestressed concrete box beam bridges with a concrete surface course provide a note similar to the following:

Calculated camber at the time of release is X inches.
Calculated camber at time of paving is X inches.
Long term camber is X inches.
Calculated deflections due to dead load applied after the beams are set (weight of surface course, railings, sidewalks, etc.) is X inches.
The vertical curve adjustment to the topping thickness at midspan is X inches upward.
The vertical curve adjustment to the topping thickness at each bearing is X inches upward/downward.
1) The concrete thickness shall be 6 inches. No variation in thickness of concrete is required.

> 2) The concrete thickness shall vary from 6 inches at each centerline of beam bearing to X inches at midspan.
> 3) The concrete thickness shall vary from X inches at each centerline of beam bearing to 6 inches at midspan.
>
> *Note to Designer: The calculated camber at the time of release is (B – C), at the time of paving is (1.8B - 1.85C), and long term is (2.45B – 2.40C). The calculated deflection due to dead load applied after the beams are set is (D + E). The vertical curve adjustment at midspan is (F) when F > 1.8B - 1.85C - D - E. The vertical curve adjustment at each bearing is (F) when F < 1.8B - 1.85C - D - E and may be upward for sag curves or downward for crest curves. Remove the reference to the vertical curve adjustment that does not apply.*
>
> *Conclude note the note above with (1), (2) or (3) as appropriate. Note (1) should be used when after placement of the topping, the top surface of the beam parallels the profile grade. Note (2) should be used when F > 1.8B - 1.85C - D - E. Note (3) should be used for all other cases.*

For composite design, show a longitudinal superstructure cross section in the plans detailing the total Topping Thickness at each centerline of bearings and at midspan. Also provide a screed elevation table.

Use the following note when the length of the box beam, measured along the grade, differs from the length, measured horizontally, by more than 3/8" [10mm]:

> NOTE TO FABRICATOR: The dimensions measured along the length of the beam, marked with a *, do not contain an allowance for the effect of the longitudinal grade. Include the proper allowance for these dimensions in the shop drawings.
>
> Note to Designer: Indicate the dimensions that require a grade adjustment with an asterisk or some other easily recognizable symbol and include that symbol in the note above.

Place the following note on the plans for prestressed concrete box beam bridges having an asphalt concrete wearing course. If the nominal thickness of intermediate course varies from the 1½" shown, revise the note accordingly. While this note specifies how to place only the two asphalt courses, the designer should recognize that two tack coat items are also required. One tack coat is applied before the intermediate asphalt concrete course. The other tack coat is applied between the intermediate and friction course.

> ASPHALT CONCRETE WEARING COURSE shall consist of a variable thickness of asphalt intermediate course, and a 1" thickness of asphalt friction course. Place the intermediate asphalt course in two operations. The first portion of the course shall be of 1½" uniform thickness. Feather the second portion of the course to place the surface parallel to and 1" below final pavement surface elevation.

Provide the following note on plans for welded attachments to steel beam or girder bridges:

> WELD ATTACHMENT of supports for concrete deck finishing machine to areas of the fascia stringer flanges designated "Compression". Do not weld attachments to areas designated "Tension". Fillet welds to compression flanges shall be at least 1" from edge of flange, be no more than 2" long, and be at least 1/4" for thicknesses up to 3/4" or 5/16" for greater than 3/4" thick.

Screed elevation tables are required for concrete decks on structural steel beam, structural steel girder, prestressed I-beam, composite box beam and other superstructure types with cast-in-place concrete decks. Screed elevations are not required for slab bridges.

In lieu of a table format, the designer may supply screed elevations through the use of a deck plan view showing elevations and stations of the required points.

In addition to the screed elevation table or diagram, provide a screed elevation note similar to the one below to define the elevations that are given. The screed elevation locations should be identified on the transverse section.

> SCREED ELEVATIONS shown represent the theoretical deck surface location prior to deflections caused by deck placement and other anticipated dead loads.

Top of haunch elevation tables are required for concrete decks on structural steel beam, structural steel girder, prestressed I-beam and other superstructure types requiring deck falsework. Top of haunch elevations are not required for slab bridges.

In addition to the top of haunch elevation table, provide a top of haunch elevation note similar to the one below to define the elevations that are given. The top of haunch elevation locations should be identified on the transverse section.

> TOP OF HAUNCH ELEVATIONS shown represent the theoretical location of the bottom of the deck above the beam/girder haunch prior to deflections caused by deck placement and other anticipated dead loads.

Final deck surface elevation tables are required for concrete decks on structural steel beam, structural steel girder, prestressed I-beam, composite box beam and other superstructure types with cast-in-place concrete decks including slab bridges.

In addition to the final deck surface elevation table, provide a final deck surface elevation note similar to the one below to define the elevations that are given.

> FINAL DECK SURFACE ELEVATIONS shown represent the deck surface location after all anticipated dead load deflections have occurred.

Use the following note for elastomeric bearings designed in accordance with LRFD 14.7.6 (i.e. Method A):

> ELASTOMERIC BEARINGS: The elastomer shall have a hardness of (50 or 60) durometer. The bearings were designed in accordance with Section 14.7.6 (Method A) of the AASHTO LRFD Bridge Design Specifications. The Long-term Compression Proof Load Test (AASHTO Standard Specifications for Highway Bridges, Division II, Section 18.7.2.6) is not required.

Use the following note for elastomeric bearings designed in accordance with LRFD 14.7.5 (i.e. Method B):

> ELASTOMERIC BEARINGS: The elastomer shall have a hardness of (50 or 60) durometer. The bearings were designed in accordance with Section 14.7.5 (Method B) of the AASHTO LRFD Bridge Design Specifications. Perform the Long-term Compression Proof Load Test in accordance with the AASHTO Standard Specifications for Highway Bridges, Division II, Section 18.7.2.6 and 18.7.4.5.

For structures that contain fracture critical components and members, place the following note in the design plans.

> FCM: All items designated FCM are Fracture Critical Members and Components and shall be furnished and fabricated according to the requirements of Section 12 of the AASHTO/AWS Bridge Welding Code D1.5.
>
> Note to Designer: Include additional wording if there exists fracture critical components such as welds, attachments, etc. that are not easily or clearly identified in the plan details. Write descriptions of such components as specific as necessary to prevent any possible confusion during fabrication.

For galvanized structures with welded shear connectors, place the following note on the same plan sheet as the shear connector spacing.

> WELDED SHEAR CONNECTORS: Install shear connectors after the decking or other walking/working surface, has been installed. Remove the galvanic coating by grinding at each connector location prior to welding.

For waterway crossing projects, include the following note on the Structure Site Plan:

> For this project, permits for Sections 401 and 404 of the Clean Water Act, are based on the limits of temporary construction fill placed in "Waters of the United States" as shown below. If either of the limits provided are exceeded by the Contractor, then a 404/401 permit modification by the Contractor will be required.
>
> Plan Area of Temporary Fill Material = X acres
> Total Volume of Temporary Fill Material = X yd^3

5.2 Final PS&E Package

The preparation of the Final Plans Specifications and Estimates Package and the Bid Ready PS&E will follow the 95% review and comment resolution meeting.

The Final Plans, Specifications and Estimate (PS&E) Plan Package to be submitted shall include the following:

- Final PS&E drawings
- Annotated copies of review comments from Progress to Advance Plan Package
- CAD Files of construction drawings
- Final Construction Cost Estimate,
- Final Construction Schedule,
- Final Special Contract Requirements
- Design Calculations (Drainage, Structural & Electrical if any)
- Load rating analysis based on LRFD methodology
- Right of Way Certification
- Environmental Permits

6.0 QUALITY CONTROL – QUALITY ASSURANCE

The Department of Public Works has established Quality Control (QC) and Quality Assurance (QA) minimum requirements and practices for ensuring that the Government's requirements and expectations are fully met. This QC/QA program applies to both the Department and to design agencies retained by the Department, and provides check and balances within the Department and the design agencies to assure quality in the final contract plans and specifications. Internal QC/QA programs of design agencies retained by the Department shall meet or exceed the guidelines in this document.

6.1 Design Team Selection

The Department will employ qualifications-based selection criteria to choose the design team for each particular bridge/highway design project, whether the team is chosen from the Department's staff or from an outside design agency.

Individuals within the Department charged with recommending and choosing the design team for each project should evaluate the qualifications of prospective designers and select a design team that has documented sufficient skills for the complexity of the particular project and has established a proven track record in bridge design.

6.2 Design

The Department requires that bridge design computations and bridge plans be made and prepared by an experienced bridge design engineer, the designer. The designer is directly responsible for the development of design calculations, drawings, specifications, and contract documents. This individual may also provide review of shop drawings related to a specific bridge design. The designer should either be registered as a Professional Structural Engineer in Guam, or should be under the direct supervision of a licensed engineer who is in responsible charge of design.

At a minimum, the design documents shall show the basis of design for all primary structural components, including superstructure, bearings, joints, and substructure components. The assumptions of the bridge design including general conditions and loadings should be documented.

It is preferable that the designer check the drawing, if the drawing was prepared by someone other than the designer. In cases where the designer is not the drawing checker, the designer must at least review the drawings to ensure that they are in conformance with the design.

The designer shall be responsible for preparing a design that follows sound engineering practice and conforms to AASHTO guides and specifications. The designer shall also be responsible for preparing an accurate and complete set of final bridge construction plans and estimated quantities.

6.3 Check

The Department requires that all bridge computations, drawings, quantities, and specifications be independently verified by an experienced engineer, the checker. The qualifications of this individual must be comparable to those of the designer.

The checker shall be responsible for ensuring correctness, constructability and completeness of the plans and calculations and adherence to pertinent specifications and manuals. The checker shall perform and prepare a set of separate, independent calculations verifying all stations, dimensions, elevations and estimated quantities.

The checker shall independently check all structural calculations to assure that the structural theory, design formulae and mathematics used by the designer are correct. The intent is not to produce two separate sets of structural calculations. However, for atypical designs, fracture critical components, and situations where the designer's theory is unclear or questionable, the checker shall perform and prepare a set of separate, independent calculations. The checker and designer shall resolve all discrepancies and the final product shall reflect mutual agreement that the design is correct.

The checker shall verify all structural calculations performed by computer analysis by preparing independent input for comparison with the designer's input. The checker shall perform an independent analysis of the output and agree with the designer on the final design.

6.4 Review

The Department requires that all bridge plans be reviewed by an experienced engineer, the reviewer. The reviewer is responsible for performing QA procedures for assuring that QC procedures have been followed. The reviewer shall be registered as a Professional Structural Engineer in Guam, and shall be the individual who seals and signs the final bridge plans and specifications.

The design agency's reviewer is responsible for the overall evaluation of the plans for completeness, consistency, continuity, constructability, general design logic and quality.

6.5 Design Documentation

Design agencies shall provide a copy of their written QA/QC program to the Department upon request.

The design agency shall perform the required checks and reviews prior to submitting prints to the Department for review. The initials of these same individuals shall be placed in the appropriate spaces in the title block signifying that they performed the work.

Design and check computations shall be kept neat and orderly so they may be easily followed and understood by a person other than the preparer.

Final plans and specifications shall be recommended by the Chief Engineer for the Department of Public Works (Highways) and shall be approved the Director of the Department of Public Works prior to receiving bids from contractors.

Revisions to plans during the construction phase shall be noted in a revision block on the drawing, shall be signed and sealed by a Professional Engineer registered in Guam and shall be recommended by the Chief Engineer for the Department of Public Works (Highways).

6.6 Construction

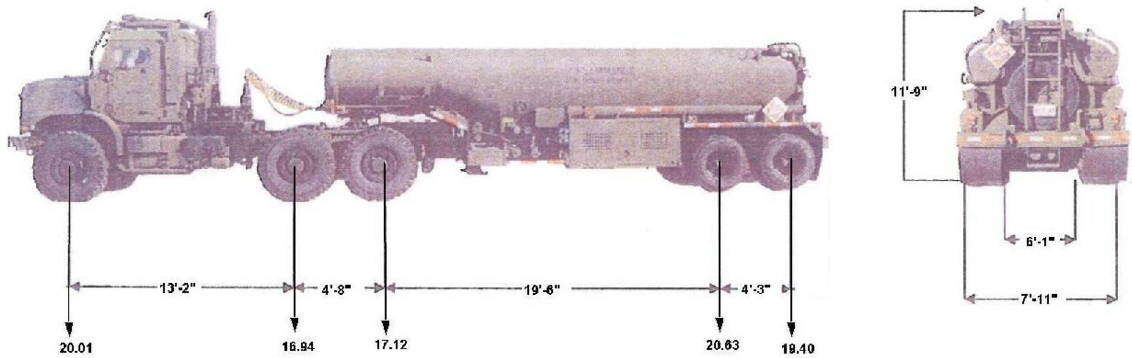
The Department will assign a Construction Management (CM) team to provide observation and inspection services during construction. The CM team may be chosen from the Department's staff or from an outside design agency, or a combination of both.

Individuals within the Department charged with recommending and choosing the CM team for each project should evaluate the qualifications of prospective construction management staff and select a design team that has documented sufficient knowledge and skills for the complexity of the particular project and has established a proven track record in construction management for bridge projects.

Specialized construction demands oversight from personnel with specialized knowledge and skills. It is recommended that observation and inspection of girder fabrication be performed by a Registered Professional Structural Engineer. It is recommended that observation and inspection of drilled shaft construction be performed by inspectors that have taken the FHWA NHI-10-017 Drilled Shaft Inspector's Qualification course.

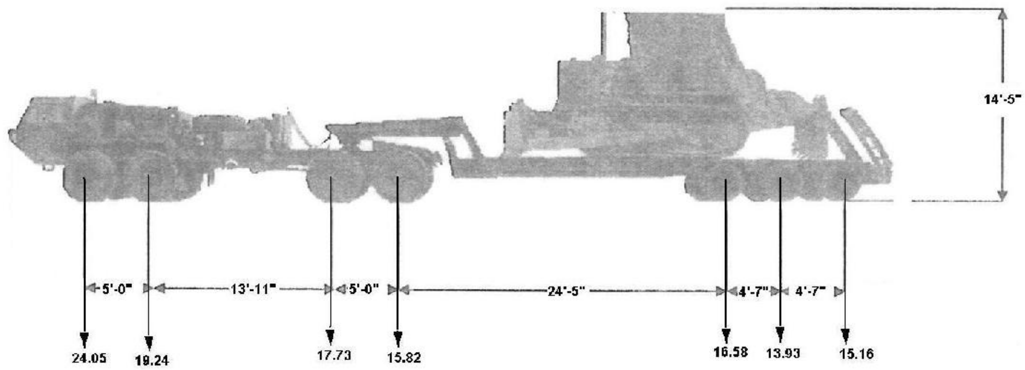
APPENDIX A

MILITARY VEHICLE CONFIGURATIONS



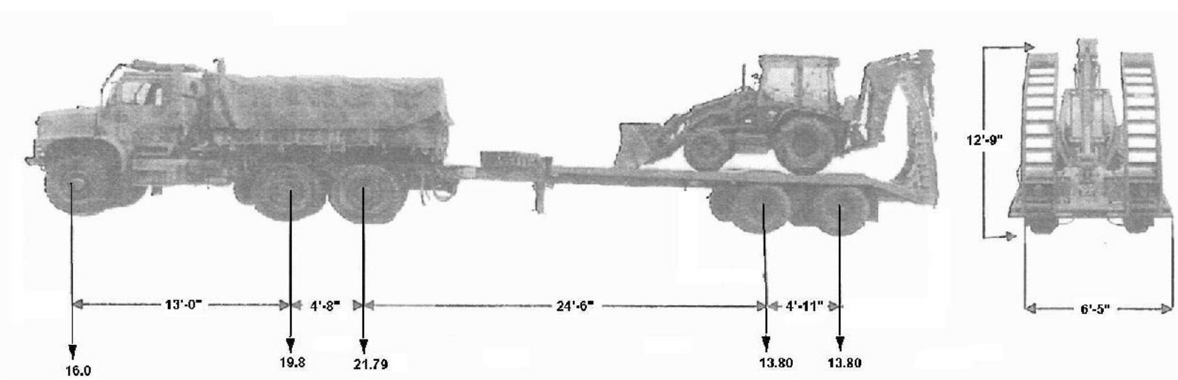
NOTE: AXLE LOADS IN KIPS

MTVR TRACTOR, SEMITRAILER REFUELER



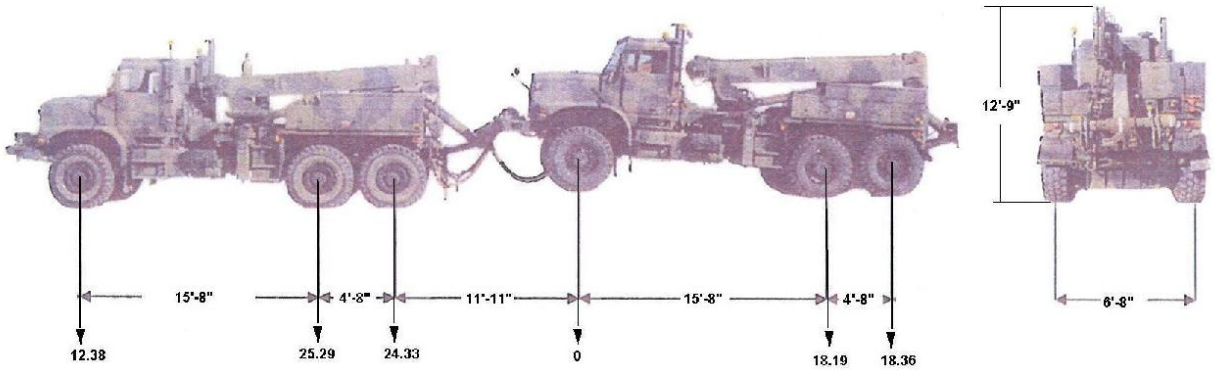
NOTE: AXLE LOADS IN KIPS

MK48 / 16 / 870 / D7LVSD7



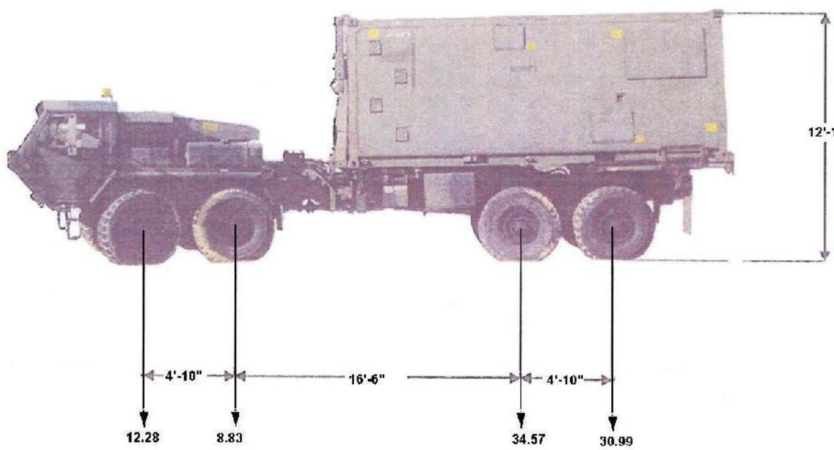
NOTE: AXLE LOAD IN KIPS

MTVR, ENGINEER EQUIPMENT TRAILER



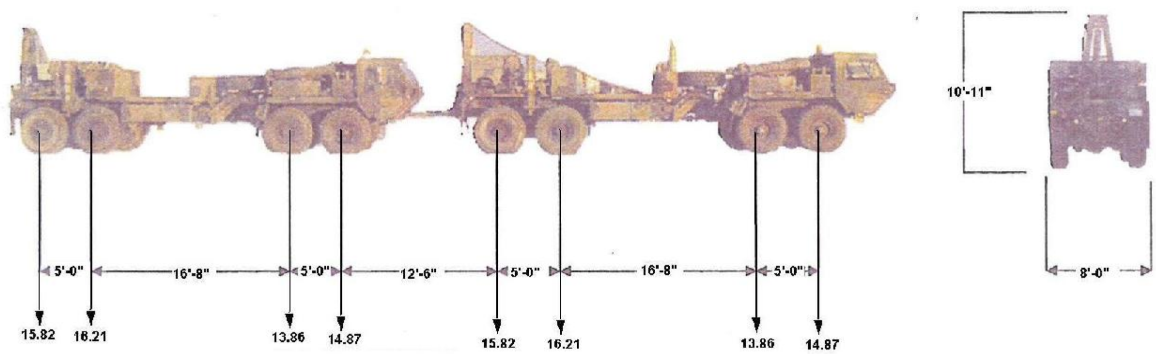
NOTE: AXLE LOADS IN KIPS

MTVR WRECKER, MTVR WRECKER



NOTE: AXLE LOADS IN KIPS

LVS MK48/18



NOTE: AXLE LOADS IN KIPS

LVS WRECKER / LVS WRECKER

APPENDIX B

COST ESTIMATING

REHABILITATION OF ROUTE 4 BRIDGES OVER THE TALOFOFO AND TOGCHA RIVERS
GU-NH-0004(015)

ENGINEER'S ESTIMATE OF PROBABLE CONSTRUCTION COST

Project Title: REHABILITATION OF ROUTE 4 BRIDGES OVER THE TALOFOFO AND TOGCHA RIVERS					
Project Number: GU-NH-0004(015)					
Location: Talofofo					
Estimate of Probable Construction Cost					
ITEM NUMBER	**DESCRIPTION**	**UNIT**	**QUANTITY**	**UNIT PRICE**	**TOTAL**
10900-0000	CONTINGENT SUM	LPSM	ALL	$ 50,000.00	$ 50,000
15101-0000	MOBILIZATION	LPSM	ALL	$130,000.00	$ 130,000
15201-0000	CONSTRUCTION SURVEY AND STAKING	LPSM	ALL	$ 52,000.00	$ 52,000
15401-0000	CONTRACTOR TESTING	LPSM	ALL	$ 26,000.00	$ 26,000
15701-0100	SOIL EROSION CONTROL, SILT FENCE	LNFT	1,140	$ 5.00	$ 5,700
15706-0200	SOIL EROSION CONTROL, CHECK DAM	EACH	1	$ 1,000.00	$ 1,000
15801-0000	WATERING FOR DUST CONTROL	MGAL	0.1	$ 95,000.00	$ 9,500
20102-0000	CLEARING AND GRUBBING	LPSM	ALL	$ 15,000.00	$ 15,000
20301-3600	REMOVAL OF RAISED PAVEMENT MARKERS	EACH	348	$ 1.00	$ 348
20302-1200	REMOVAL OF GUARDRAIL	LNFT	1,126	$ 10.00	$ 11,260
20302-2600	REMOVAL OF PAVEMENT MARKINGS	LNFT	228	$ 0.75	$ 171
20303-0100	REMOVAL OF APPROACH SLAB	SQYD	646	$ 63.00	$ 40,698
20303-1600	REMOVAL OF PAVEMENT, ASPHALT	SQYD	412	$ 10.00	$ 4,120
20304-1000	REMOVAL OF STRUCTURES AND OBSTRUCTIONS	LPSM	ALL	$ 5,000.00	$ 5,000
20304-4000	REMOVAL OF BRIDGE SUPERSTRUCTURE	LPSM	ALL	$300,000.00	$ 300,000
20304-7000	REMOVAL OF UTILITY CONDUITS	LPSM	ALL	$ 13,000.00	$ 13,000
20315-0000	SAWCUTTING PAVEMENT	LNFT	172	$ 3.00	$ 516
20401-0000	ROADWAY EXCAVATION	CUYD	61	$ 20.00	$ 1,220
20420-0000	EMBANKMENT CONSTRUCTION	CUYD	64	$ 35.00	$ 2,240
20801-0000	STRUCTURE EXCAVATION	CUYD	82	$ 25.00	$ 2,050
20803-0000	STRUCTURAL BACKFILL	CUYD	69	$ 40.00	$ 2,760
20811-0000	SHORING AND BRACING	SQFT	500	$ 40.00	$ 20,000
25301-0000	GABIONS	SQFT	270	$ 150.00	$ 40,500
30112-0100	AGGREGATE SHOULDER	CUYD	10	$ 65.00	$ 650
40301-0410	HOT ASPHALT CONCRETE FRICTION COURSE, GRADING D, 1 INCH	SQYD	1,526	$ 22.50	$ 34,335
41202-0000	TACK COAT	GAL	155	$ 10.00	$ 1,550
55201-0100	STRUCTURAL CONCRETE, CLASS A	CUYD	1,022	$ 550.00	$ 562,100
55205-0000	REPAIR CONCRETE	SQYD	50	$ 300.00	$ 15,000
55301-3300	PRECAST, PRESTRESSED CONCRETE AASHTO GIRDER	EACH	29	$ 18,000.00	$ 522,000
55401-2000	REINFORCING STEEL, EPOXY COATED	LB	166,500	$ 1.25	$ 208,125
55601-0900	BRIDGE RAILING, STEEL	LNFT	670	$ 100.00	$ 67,000
56401-1000	BEARING DEVICE, ELASTOMERIC	EACH	58	$ 500.00	$ 29,000
61101-2000	PERMANENT RELOCATION OF 12" WATERLINE, INCLUDING FITTINGS	LPSM	ALL	$ 90,000.00	$ 90,000
61601-1100	SLOPE PAVING, CONCRETE, REMOVE AND REPLACE	LPSM	ALL	$ 5,000.00	$ 5,000
61701-5010	GUARDRAIL, TYPE "W"	LNFT	553	$ 47.00	$ 25,991
61702-0010	GUARDRAIL ANCHORAGE, APPROACH END	EACH	6	$ 5,650.00	$ 33,900
61702-0020	GUARDRAIL ANCHORAGE, TRAILING END	EACH	1	$ 3,500.00	$ 3,500
61707-0000	STRUCTURE TRANSITION RAILING	LNFT	168	$ 62.00	$ 10,416
61807-0000	CONCRETE PARAPET	LNFT	670	$ 50.00	$ 33,500
62101-0200	RESET SURVEY MONUMENT	EACH	1	$ 5,000.00	$ 5,000
62502-0000	TURF ESTABLISHMENT	SQYD	525	$ 16.00	$ 8,400
62516-3000	MULCHING, HYDRAULIC METHOD, BONDED FIBER MATRIX	SQYD	600	$ 5.00	$ 3,000
62701-0100	SOD, STRIP, REINFORCED BIOSWALE	SQYD	89	$ 30.00	$ 2,670
63316-1000	REMOVE AND RESET SIGN	EACH	2	$ 300.00	$ 600
63401-1501	PAVEMENT MARKING, TYPE H, 4" WIDE, SOLID LINE, WHITE	LNFT	717	$ 2.10	$ 1,505
63401-1503	PAVEMENT MARKING, TYPE H, 4" WIDE, SOLID LINE, YELLOW	LNFT	716	$ 2.10	$ 1,504
63406-0001	RAISED PAVEMENT MARKER, REFLECTORIZED 2-WAYS, "YY"	EACH	1,077	$ 6.00	$ 6,462
63501-0000	TEMPORARY TRAFFIC CONTROL	LPSM	ALL	$ 85,000.00	$ 85,000
63501-1000	TEMPORARY TRAFFIC CONTROL, TRAFFIC AND SAFETY SUPERVISOR	LPSM	ALL	$ 73,027.00	$ 73,027
63610-2800	CONDUIT, 4-INCH, PVC	LNFT	2,800	$ 7.00	$ 19,600
63701-0100	FIELD OFFICE	LPSM	ALL	$ 10,000.00	$ 10,000
				SUBTOTAL	**$ 2,591,917**
				GRT (4.17%)	**$ 108,083**
				TOTAL	**$ 2,700,000**

NCHRP
REPORT 483

NATIONAL
COOPERATIVE
HIGHWAY
RESEARCH
PROGRAM

Bridge Life-Cycle Cost Analysis

TRANSPORTATION RESEARCH BOARD
OF THE NATIONAL ACADEMIES

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

NCHRP REPORT 483

Bridge Life-Cycle Cost Analysis

HUGH HAWK
National Engineering Technology Corporation
Arlington Heights, IL

SUBJECT AREAS
Bridges, Other Structures, and Hydraulics and Hydrology

Research Sponsored by the American Association of State Highway and Transportation Officials
in Cooperation with the Federal Highway Administration

TRANSPORTATION RESEARCH BOARD

WASHINGTON, D.C.
2003
www.TRB.org

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

In recognition of these needs, the highway administrators of the American Association of State Highway and Transportation Officials initiated in 1962 an objective national highway research program employing modern scientific techniques. This program is supported on a continuing basis by funds from participating member states of the Association and it receives the full cooperation and support of the Federal Highway Administration, United States Department of Transportation.

The Transportation Research Board of the National Academies was requested by the Association to administer the research program because of the Board's recognized objectivity and understanding of modern research practices. The Board is uniquely suited for this purpose as it maintains an extensive committee structure from which authorities on any highway transportation subject may be drawn; it possesses avenues of communications and cooperation with federal, state and local governmental agencies, universities, and industry; its relationship to the National Research Council is an insurance of objectivity; it maintains a full-time research correlation staff of specialists in highway transportation matters to bring the findings of research directly to those who are in a position to use them.

The program is developed on the basis of research needs identified by chief administrators of the highway and transportation departments and by committees of AASHTO. Each year, specific areas of research needs to be included in the program are proposed to the National Research Council and the Board by the American Association of State Highway and Transportation Officials. Research projects to fulfill these needs are defined by the Board, and qualified research agencies are selected from those that have submitted proposals. Administration and surveillance of research contracts are the responsibilities of the National Research Council and the Transportation Research Board.

The needs for highway research are many, and the National Cooperative Highway Research Program can make significant contributions to the solution of highway transportation problems of mutual concern to many responsible groups. The program, however, is intended to complement rather than to substitute for or duplicate other highway research programs.

Note: The Transportation Research Board of the National Academies, the National Research Council, the Federal Highway Administration, the American Association of State Highway and Transportation Officials, and the individual states participating in the National Cooperative Highway Research Program do not endorse products or manufacturers. Trade or manufacturers' names appear herein solely because they are considered essential to the object of this report.

NCHRP REPORT 483

Project C12-43 FY'96

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NOTICE

The project that is the subject of this report was a part of the National Cooperative Highway Research Program conducted by the Transportation Research Board with the approval of the Governing Board of the National Research Council. Such approval reflects the Governing Board's judgment that the program concerned is of national importance and appropriate with respect to both the purposes and resources of the National Research Council.

The members of the technical committee selected to monitor this project and to review this report were chosen for recognized scholarly competence and with due consideration for the balance of disciplines appropriate to the project. The opinions and conclusions expressed or implied are those of the research agency that performed the research, and, while they have been accepted as appropriate by the technical committee, they are not necessarily those of the Transportation Research Board, the National Research Council, the American Association of State Highway and Transportation Officials, or the Federal Highway Administration, U.S. Department of Transportation.

Each report is reviewed and accepted for publication by the technical committee according to procedures established and monitored by the Transportation Research Board Executive Committee and the Governing Board of the National Research Council.

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Advisers to the Nation on Science, Engineering, and Medicine

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www.national-academies.org

AUTHOR ACKNOWLEDGMENTS

The research reported herein was performed under NCHRP Project 12-43 by the National Engineering Technology Corporation (NET Corp.) and Delcan Corporation. NET Corp. was the contractor for this study and for the software development.

Hugh R. Hawk, Chief Bridge Engineer, Delcan Corporation, was the principal investigator. The other authors of this report are Dr. Andy Lemer of the Matrix Group and Dr. Kumares Sinha, Professor of Civil Engineering, Purdue University. The bulk of the software programming was conducted by Nimira Kurji, Delcan Corporation. Assistance in the testing of the software was provided by Stepanka Elias, a former employee of Delcan and now a research assistant at the University of Toronto. The bulk of the work was done under the direct supervision of Hugh Hawk. Background material was coordinated by Dr. Sinha, and the State-of-the-Art Study was conducted by Dr. Lemer.

FOREWORD

By David B. Beal
Staff Officer
Transportation Research Board

This report contains the findings of a study to develop a methodology for bridge life-cycle cost analysis (BLCCA) for use by transportation agencies. The report describes the research effort leading to the recommended methodology and includes a guidance manual for carrying out BLCCA and software that automates the methodology. The material in this report will be of immediate interest to engineers concerned with the life-cycle cost analysis of major bridges.

Transportation officials consider life-cycle cost analysis an important technique for assisting with investment decisions. Several recent legislative and regulatory initiatives recognize the potential benefits of life-cycle cost analysis and call for consideration of such analyses for infrastructure investments, including investments in highway bridge programs. Because a commonly accepted, comprehensive methodology for bridge life-cycle cost analysis (BLCCA) did not exist, NCHRP Project 12-43 was initiated.

Under NCHRP Project 12-43, National Engineering Technology Corporation developed a comprehensive procedure for life-cycle cost analysis. Of particular note is the explicit introduction of vulnerability and uncertainty in the analysis. Consideration of vulnerability and uncertainty results in a more realistic estimate of life-cycle cost. Although default values are provided for cost parameters, users will benefit from the development and use of parameters specific to the structure and environment in question.

The proposed methodology is fully described in the Guidance Manual (Part II of the report). The methodology is implemented in software contained on a CD bound with the report (*CRP-CD-26*). The report appendixes, the Guidance Manual, and a User's Manual are accessible from the software. The User's Manual presents four examples of the application of the methodology.

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BRIDGE LIFE-CYCLE COST ANALYSIS

SUMMARY

NCHRP Project 12-43, "Life-Cycle Cost Analysis for Bridges," has resulted in *NCHRP Report 483* and *CRP-CD-26*, which can be used by professionals to undertake life-cycle costing analysis for bridges. The report has two parts. Part I (the Report) establishes guidelines and standardizes procedures for conducting life-cycle costing. Part II (the Guidance Manual) is useful to all professionals engaged in life-cycle cost analysis either for the repair of existing structures or for the evaluation of new bridge alternatives. The Guidance Manual outlines the concept of life-cycle costing, identifies sources for data, and explains the methodology by which life-cycle costing can be conducted.

CRP-CD-26 contains the appendixes to the Report (Appendixes A, B, D, and E; Appendix C is the Guidance Manual); the User's Manual and Guidance Manual both as Word documents and in portable document format (pdf); and the bridge life-cycle cost analysis (BLCCA) software. The BLCCA software provides a tool for professionals to apply the life-cycle cost-analysis concepts and methodologies to the analysis of bridges. The software considers agency and user costs and enables the user to consider both vulnerability and uncertainty in the analysis.

In combination, the Report, Guidance Manual, and software are a powerful tool that can be applied to the decision-making process for the repair or selection of cost-effective alternatives for the preservation of bridge assets for short-term and long-term planning horizons. *NCHRP Report 483* and *CRP-CD-26* are companions to the network-based Bridge Management Systems.

APPENDIX C

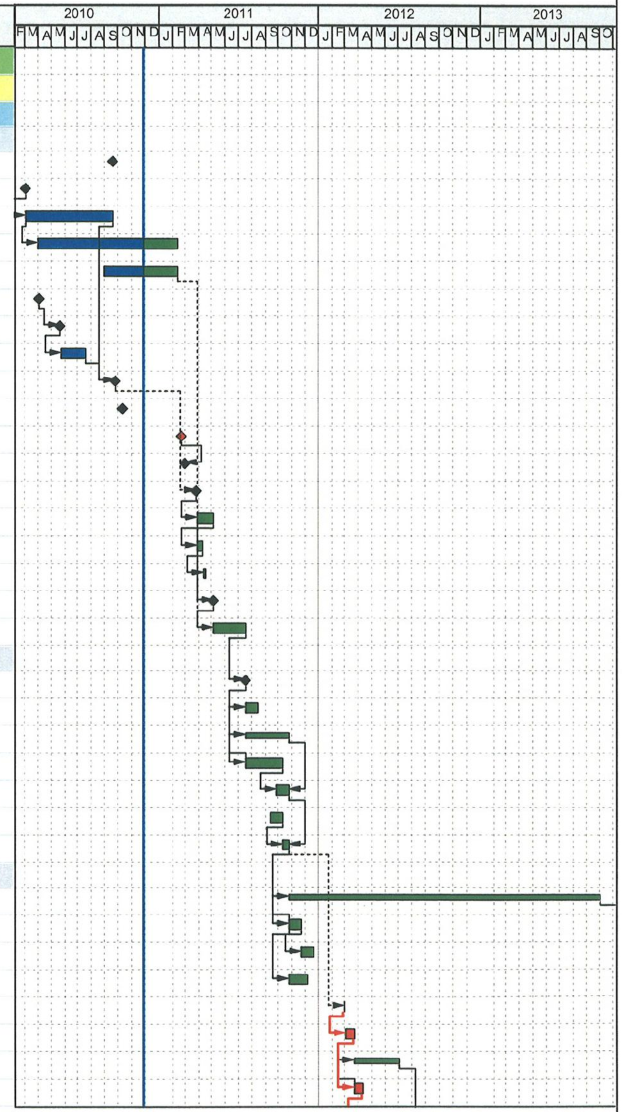
SAMPLE CONSTRUCTION SCHEDULE

Guam Department of Public Works

GUAM CONTRACTOR

Data Date : 30-Nov-10
Print Date : 01-Feb-11

Activity ID	Activity Name	Original Duration	Start	Finish	CAL	Predecessors	Relationship
Working Draft		1192	03-Mar-10 A	03-Oct-13			
GU-DAR-0001(014)		1192	03-Mar-10 A	03-Oct-13			
Route 1/Route 8 Intersection Improvements and Agana Bridges Replacement [P1006]		1192	03-Mar-10 A	03-Oct-13			
Procurement		384	03-Mar-10 A	18-Jul-11			
A1000	Record of Decision (ROD) - Department of Defense (DoD)	0		20-Sep-10 A	03...		
A1010	NTP - Preliminary Design	0	03-Mar-10 A		03...		
A1020	Preliminary Design	140	03-Mar-10 A	20-Sep-10 A	03...	A1010	
A1030	Environmental Permit	205	01-Apr-10 A	14-Feb-11	03...	A1020	SS
A1040	Record of Decision (ROD) - FHWA [NEPA Clearance]	100	31-Aug-10 A	14-Feb-11	03...		
A1100	Issue RFQ	0	02-Apr-10 A		03...		
A1110	Statement of Qualifications Due	0		21-May-10 A	03...	A1100	
A1120	Shortlist Firms	40	24-May-10 A	20-Jul-10 A	03...	A1110	
A1200	Issue RFP	0	24-Sep-10 A		03...	A1020, A1120	
A1202	Mandatory Pre-Proposal Conference	0	12-Oct-10 A		03...		
A1204	Deadline for Submitting Proposers' Questions	0		22-Feb-11*	03...		
A1206	Deadline for DPW Response to Proposers' Questions	0		01-Mar-11	03...	A1204	FF+5
A1210	Proposal Due	0		28-Mar-11*	03...	A1200	
A1220	Evaluation / Selection	28	29-Mar-11	05-May-11	03...	A1210	
A1222	DPW Requests for Information (RFIs) to Proposers	10	29-Mar-11	11-Apr-11	03...	A1220	SS
A1224	Proposer Responses to RFIs	5	12-Apr-11	18-Apr-11	03...	A1222	
A1230	Announce Best Value Proposer	0		05-May-11	03...	A1220	
A1240	Contract Approval (GovGuam) and Award - Design-Build	50	06-May-11	18-Jul-11	03...	A1040, A1230	
Design		98	19-Jul-11	24-Oct-11			
A5000	NTP - Design-Build	0	19-Jul-11		01...	A1240	
A5010	Mobilization	28	19-Jul-11	15-Aug-11	01...	A5000	
A5100	FINAL DESIGN	98	19-Jul-11	24-Oct-11	01...	A5110	SS
A5110	Bridge Design	84	19-Jul-11	10-Oct-11	01...	A5000	SS
A5120	Bridge Design Approval	28	27-Sep-11	24-Oct-11	01...	A5100, A5110	FF, FS-14
A5130	Shop Drawing Preparation	28	13-Sep-11	10-Oct-11	01...		
A5140	Shop Drawing Approval	14	11-Oct-11	24-Oct-11	01...	A5130, A5120	FS, FF
Construction		710	25-Oct-11	03-Oct-13			
A6000	CONSTRUCTION	710	25-Oct-11	03-Oct-13	01...	A6010	SS
A6010	Bridge Girder Fabrication	28	25-Oct-11	21-Nov-11	01...	A5140	
A6020	Bridge Girder Curing	28	22-Nov-11	19-Dec-11	01...	A6010	
A6030	Reinforcing Steel Fabrication	42	25-Oct-11	05-Dec-11	01...	A6010	SS
A6040	Set-up Traffic Control	1	01-Mar-12*	01-Mar-12	01...	A5140	
A6100	PHASE 1 - Median Removal and Temporary Pavement	21	02-Mar-12	22-Mar-12	01...	A6040	
A6200	PHASE 2 - North Section of Bridge and Related Roadway	101	23-Mar-12	01-Jul-12	01...	A6202	SS
A6202	Demolish Portion of Existing Bridge	21	23-Mar-12	12-Apr-12	01...	A6100	

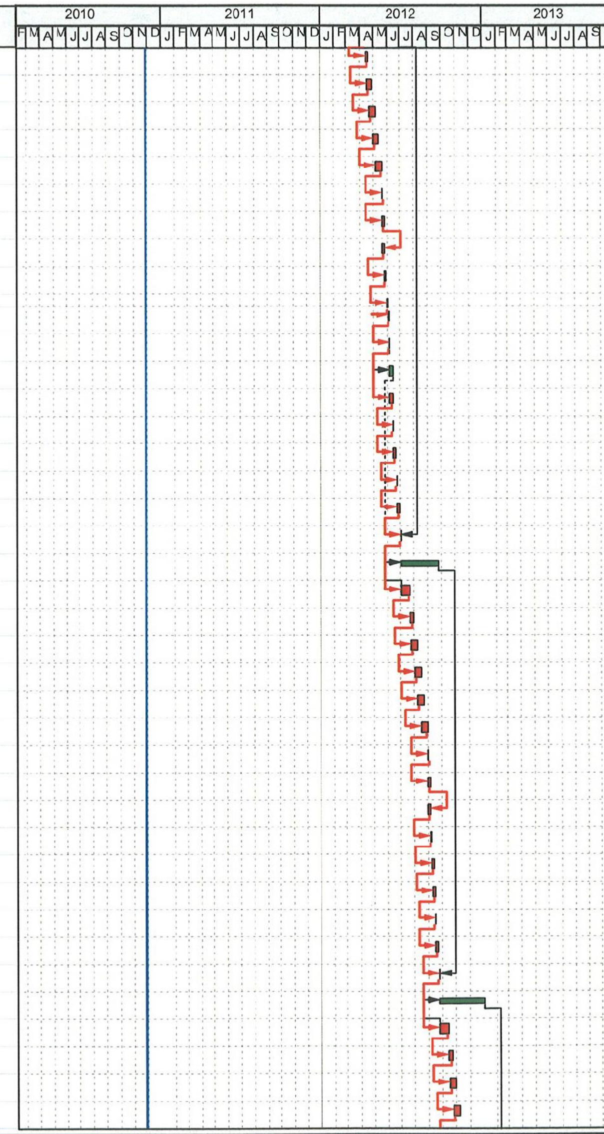


Actual Work — Critical Remaining Work — Critical Milestone
Remaining Work — Milestone — Actual Level of Effort

Guam DPW Projects
[Single] Detailed Project Schedule

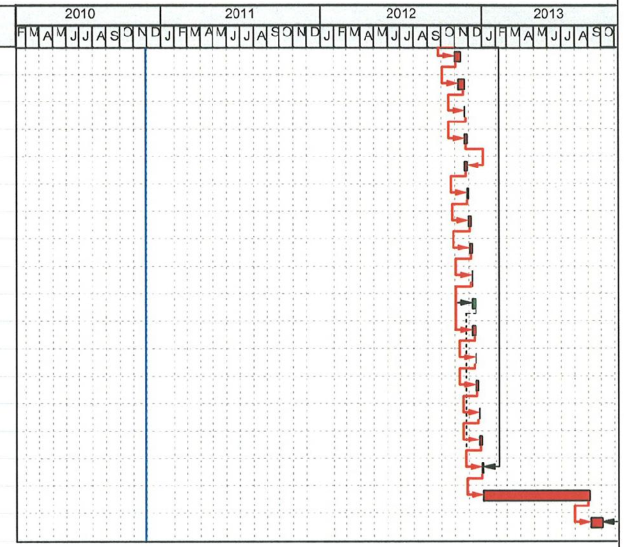
TASK filter: All Activities

Activity ID	Activity Name	Original Duration	Start	Finish	CAL	Predecessors	Relationship
A6204	Drive Sheet Piling	7	13-Apr-12	19-Apr-12	01...	A6202	
A6206	Excavate for Abutments and Piers	14	16-Apr-12	29-Apr-12	01...	A6204	FS-4
A6208	Drive Piles for Abutments and Piers	14	23-Apr-12	06-May-12	01...	A6206	SS+7
A6210	Form Abutment and Pier Caps	14	30-Apr-12	13-May-12	01...	A6208	SS+7
A6212	Tie Reinforcing Steel	14	07-May-12	20-May-12	01...	A6210	SS+7
A6214	Place Concrete for Abutment and Pier Caps	1	21-May-12	21-May-12	01...	A6212	
A6216	Concrete Curing - High Early Strength	7	21-May-12	27-May-12	01...	A6214	SS
A6218	Strip Forms	7	21-May-12	27-May-12	01...	A6216	FF
A6220	Erect Girders	3	28-May-12	30-May-12	01...	A6218	
A6222	Form Deck Overlay	5	31-May-12	04-Jun-12	01...	A6220	
A6224	Tie Reinforcing Steel	4	03-Jun-12	06-Jun-12	01...	A6222	FS-2
A6226	Place Concrete for Deck Overlay	1	07-Jun-12	07-Jun-12	01...	A6224	
A6227	Concrete Curing - High Early Strength	7	08-Jun-12	14-Jun-12	01...	A6226	
A6228	Form Sidewalk and Tie Reinforcing Steel	7	08-Jun-12	14-Jun-12	01...	A6226	
A6230	Place Concrete for Sidewalk	1	15-Jun-12	15-Jun-12	01...	A6228	
A6232	Form Barrier and Tie Reinforcing Steel	7	16-Jun-12	22-Jun-12	01...	A6230	
A6234	Place Concrete for Barrier	1	23-Jun-12	23-Jun-12	01...	A6232	
A6236	Install Steel Railing, Strip Forms, Rub Concrete	7	24-Jun-12	30-Jun-12	01...	A6234	
A6238	Install Portable Concrete Barriers for Next Phase	1	01-Jul-12	01-Jul-12	01...	A6236, A6200, A6227	FS, FF, FS
A6300	PHASE 3 - Middle Section of Bridge and Related Roadway	85	02-Jul-12	24-Sep-12	01...	A6302	SS
A6302	Demolish Portion of Existing Bridge	21	02-Jul-12	22-Jul-12	01...	A6238	
A6304	Drive Sheet Piling	7	23-Jul-12	29-Jul-12	01...	A6302	
A6306	Excavate for Abutments and Piers	14	26-Jul-12	08-Aug-12	01...	A6304	FS-4
A6308	Drive Piles for Abutments and Piers	14	02-Aug-12	15-Aug-12	01...	A6306	SS+7
A6310	Form Abutment and Pier Caps	14	09-Aug-12	22-Aug-12	01...	A6308	SS+7
A6312	Tie Reinforcing Steel	14	16-Aug-12	29-Aug-12	01...	A6310	SS+7
A6314	Place Concrete for Abutment and Pier Caps	1	30-Aug-12	30-Aug-12	01...	A6312	
A6316	Concrete Curing - High Early Strength	7	30-Aug-12	05-Sep-12	01...	A6314	SS
A6318	Strip Forms	7	30-Aug-12	05-Sep-12	01...	A6316	FF
A6320	Erect Girders	3	06-Sep-12	08-Sep-12	01...	A6318	
A6322	Form Deck Overlay	5	09-Sep-12	13-Sep-12	01...	A6320	
A6324	Tie Reinforcing Steel	4	12-Sep-12	15-Sep-12	01...	A6322	FS-2
A6326	Place Concrete for Deck Overlay	1	16-Sep-12	16-Sep-12	01...	A6324	
A6327	Concrete Curing - High Early Strength	7	17-Sep-12	23-Sep-12	01...	A6326	
A6338	Install Portable Concrete Barriers for Next Phase	1	24-Sep-12	24-Sep-12	01...	A6300, A6327	FF, FS
A6400	PHASE 4 - South Section of Bridge and Related Roadway	101	25-Sep-12	03-Jan-13	01...	A6402	SS
A6402	Demolish Portion of Existing Bridge	21	25-Sep-12	15-Oct-12	01...	A6338	
A6404	Drive Sheet Piling	7	16-Oct-12	22-Oct-12	01...	A6402	
A6406	Excavate for Abutments and Piers	14	19-Oct-12	01-Nov-12	01...	A6404	FS-4
A6408	Drive Piles for Abutments and Piers	14	26-Oct-12	08-Nov-12	01...	A6406	SS+7



Actual Work — Critical Remaining Work ◆ — ◆ Critical Milestone
Remaining Work ◆ — ◆ Milestone — Actual Level of Effort

Activity ID	Activity Name	Original Duration	Start	Finish	CAL	Predecessors	Relationship
A6410	Form Abutment and Pier Caps	14	02-Nov-12	15-Nov-12	01...	A6408	SS+7
A6412	Tie Reinforcing Steel	14	09-Nov-12	22-Nov-12	01...	A6410	SS+7
A6414	Place Concrete for Abutment and Pier Caps	1	23-Nov-12	23-Nov-12	01...	A6412	
A6416	Concrete Curing - High Early Strength	7	23-Nov-12	29-Nov-12	01...	A6414	SS
A6418	Strip Forms	7	23-Nov-12	29-Nov-12	01...	A6416	FF
A6420	Erect Girders	3	30-Nov-12	02-Dec-12	01...	A6418	
A6422	Form Deck Overlay	5	03-Dec-12	07-Dec-12	01...	A6420	
A6424	Tie Reinforcing Steel	4	06-Dec-12	09-Dec-12	01...	A6422	FS-2
A6426	Place Concrete for Deck Overlay	1	10-Dec-12	10-Dec-12	01...	A6424	
A6427	Concrete Curing - High Early Strength	7	11-Dec-12	17-Dec-12	01...	A6426	
A6428	Form Sidewalk and Tie Reinforcing Steel	7	11-Dec-12	17-Dec-12	01...	A6426	
A6430	Place Concrete for Sidewalk	1	18-Dec-12	18-Dec-12	01...	A6428	
A6432	Form Barrier and Tie Reinforcing Steel	7	19-Dec-12	25-Dec-12	01...	A6430	
A6434	Place Concrete for Barrier	1	26-Dec-12	26-Dec-12	01...	A6432	
A6436	Install Steel Railing, Strip Forms, Rub Concrete	7	27-Dec-12	02-Jan-13	01...	A6434	
A6438	Remove Portable Concrete Barriers	1	03-Jan-13	03-Jan-13	01...	A6436, A6400, A6427	FS, FF, FS
A6500	PHASE 5 - Chalan Santo Papa Juan Pablo Dos	245	04-Jan-13	05-Sep-13	01...	A6438	
A6900	Demobilization and Final Acceptance	28	06-Sep-13	03-Oct-13	01...	A6000, A6500	FF, FS



Actual Work | Critical Remaining Work | Critical Milestone
Remaining Work | Milestone | Actual Level of Effort

APPENDIX D

HYDRAULIC DESIGN REFERENCE DOCUMENTS

ATTACHMENT V

GUAM COMPREHENSIVE STUDY FOR WATER AND RELATED LAND RESOURCES

Agana Bay Typhoon and Storm-Surge Protection Study

US Army Corps of Engineers
Honolulu District

TECHNICAL DOCUMENTATION

January 1984

TABLE 5. TYPHOON-GENERATED STORM-SURGE INUNDATION LEVELS (MSL)

Storm Frequency	Surge Level MLLW*	Surge Level MSL*	Breaking Wave Height (Ft)	Total Inundation (Ft)
10-year event (10%)	+6.7	+5.3	3.1	+8.4
50-year event (2%)	+8.8	+7.4	4.0	+11.4
100-year event (1%)	+9.4	+8.0	4.3	+12.3
500-year event (0.2%)	+10.1	+8.7	4.6	+13.3

* MLLW (Mean Lower Low Water); MSL(Mean Sea Level)

(4) The significance of the inundation levels lies in the existing relatively low elevation of the study ara. The old community of Anigua lies generally between 6 and 10 feet above mean sea level (MSL) divided by Marine Drive which ranges from about 8 feet to 13 feet, east to west. Downtown Agana lies at elevations between 6.5 to 11 feet above MSL. Marine Drive fronting Agana ranges between 6.5 to 7.5 feet and 11.2 feet at Agana River bridge. The East Agana waterfront strip is about 8.5 feet (MSL) and Marine Drive about 1 foot higher. Western Tamuning or Apurguan averages about 6 to 8 feet in elevation (MSL) and the shorefront area at Dungca's Beach ranges between 3.5 to 10 feet. For the most part, there are few areas higher than 10-11 feet (MSL) in the coastal plain. Ninety-foot high cliffs are in back of Anigua and eastern Agana, and almost immediately inland of Marine Drive along the East Agana strip and Trinchera Beach. There is also an undeveloped 15-foot incline behind the Dungca's Beach residences sloping up to Camp Watkins Road. Thus, most of the Agana Bay coastal plain lies within flood levels of a 50-year storm-surge event, which could flood areas up to about 11.4 feet (MSL) according to the above calculations. Without wave runup, the surge alone would flood areas up to an elevation of 8.8 feet (MSL) (see Table 5).

(5)

(a) There are three flood zones designated on Figure 7. All indicate the areal extent of flooding from a 100-year typhoon-generated storm-surge flood event. These (and other) zones are designated under the National Flood Insurance Program to indicate degrees of flood risk, and together with other factors, are used to assign actuarial insurance rates to structures and contents insured under that program. The 100-year or 1 percent flood is the standard level of flooding used in the program..

(b) The narrow zone near the shoreline is called the "V" or Velocity Zone which is the inland extent of a 3-foot breaking wave where the effective water depth during the 100-year flood decreases to less than 4 feet. The V-zone is the Special Flood Hazard Area where velocity hazards to structures could occur. It is determined by approximate methods.

(c) The large "A" Zone is a Special Flood Hazard Area inundated by the 100-year flood. Structures in the A Zone are not subject to wave action, but residual forward movement of the breaking wave may be present. The A-zone is designated an A3 zone, which indicates that the difference in flood level between a 100-year (1%) and 10-year (10%) storm-surge flood event averages 1.5 feet. This is not an empirically derived difference in flood elevations.

Coastal Bridges and Design Storm Frequency

Introduction

The state of practice for assigning design frequency for coastal bridges assumes that the coastal environment behaves hydraulically similarly to riverine environments. In reality, bridges in a coastal environment have quite different hydraulic behavior and may need application of alternative design elements and frequencies.

The coastal environment includes the Atlantic, Gulf, Caribbean, Pacific, Great Lakes, and other larger estuaries and water bodies. While the same general mechanisms apply to these areas, each also may present the designer with unique hydraulic concerns and constraints. FHWA has already developed coastal hydraulic guidance in the HEC-25 document and is in the process of updating and enhancing this HEC-25 document into a second edition.

Background

The current state-of-practice for determining design frequencies in the hydraulic analysis and design of bridges assume the presence of riverine flow characteristics. Riverine floods exhibit unsteady flow (hydrograph) with channel attenuation of the flood event. The water surface elevation will increase until reaching the hydrograph peak, and then the flood will recede (therefore exhibiting a time dependency of flow). As a matter of practice, hydraulic engineers treat riverine flood event as a steady flow scenario with the peak of the flood hydrograph representing the design discharge. The advantages of this steady flow peak flow design approach are:

- data collection, validation, and computational requirements and limitations often preclude accurately simulating the unsteady flow.
- by focusing on the peak flow, the hydraulic design has some redundancy during lesser discharges that occur during most of the flood event.
- determining the design frequency can focus on a single probabilistic element (flow), rather than a multitude of elements (flow, flood duration, boundary conditions, etc).

FHWA regulates hydraulic structures under 23 CFR 650, subpart A (650.A): “Location and Hydraulic Design of Encroachments on Flood Plains.” Regulatory practices codify the steady state peak flow design approach by requiring analyses such as the FHWA Location Hydraulic Studies (23 CFR 650.111), Design Hydraulic Studies (23 CFR 650.115), and FEMA Flood Insurance Studies (FIS) to support the National Flood Insurance Program (44 CFR 60.3).

In the coastal environment, design practice assumes that flood events would essentially behave in a manner similar to the riverine environment. However, bridge failure mechanisms associated with recent storm events have resulted in a reevaluation of these assumptions. The result is a need to differentiate how FHWA considers the state-of-practice to hydraulically design bridges in the coastal environment.

Coastal Hydraulic Characteristics

As illustrated in Figure 1, the coastal flood event may consist of several hydraulic constituents that must be considered above the local mean water level, including,

- ***Astronomical tides***: the water surface variations primarily resulting from gravitational interactions of the earth, moon, and sun.
- ***Storm Surge***: the difference in mean water level from the astronomical tide as a result of the storm. This includes all storm-related increases such as those due to atmospheric pressure, wave setup (the increase in mean water level due to the presence of waves which can be significant in a surf zone), and wind stress across the continental shelf and across enclosed bodies of water (this can be the dominant effect in shallow, broad estuaries). Note that in design this is often called the "design still-water level" since it is the mean water level averaged over several minutes to remove the short-period (5-15 seconds) wave fluctuations.
- ***Wave Crest elevation***: The additional elevation above the design still water level (SWL) that the highest portions of the waves will reach (variable Y). This is typically assumed as roughly 75% of the wave height (variable H) for conservative design.

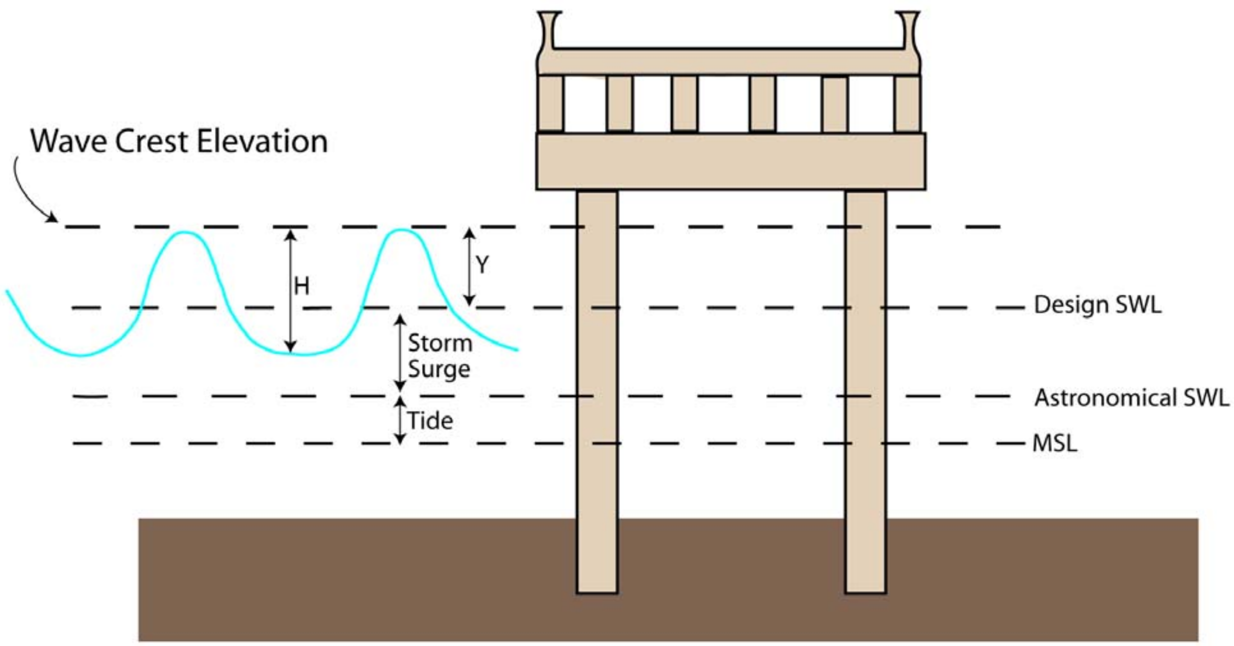


Figure 1. Characteristics of Coastal Hydraulics (Douglass, 2005)

The recent hurricane events striking the Gulf Coast caused damages to bridges primarily through the combination of the storm surge and wave crest elevation constituents. Forensic analysis after Ivan and initial site inspections after Katrina indicate that the bridges were damaged by waves. The waves only reached the bridge deck elevations because of the storm surge. Very preliminary calculations imply that the horizontal wave slamming (impact) loads combined with vertical wave uplift loads to progressively "bump" the decks off the pile caps. The individual "bumping" loads were a result from individual waves. In every case, the storm surge put the "still-water level" near the low chord elevation for enough time for waves to move the decks.

Typically FEMA coastal FIS's provide both the storm surge (stillwater) and wave crest elevation values for the 50-, 100-, and 500-year frequencies. Additionally, they provide some indication of flow velocities and hazards.

The wave height of interest may not be the maximum wave generated by a storm event. A design may use the significant wave height or other approach (e.g., the one-percent or two-percent highest wave). Such distinctions recognize that a bridge may be able to withstand wave uplift and impact forces from a small number of the very largest waves, but fail from the repetitive impact/uplift cycles of smaller waves in a storm sea state.

Typical State DOT Bridge Design Frequencies

Most State DOTs design their bridges to a minimum of a two percent exceedance flow frequency (50-year storm event)[1]. The design frequencies are codified by:

- State drainage manuals (approved by the respective FHWA Division offices).
- AASHTO: 23 CFR 625.4(a)(1) and 625.4(a)(2).
- FHWA Floodplain Regulations 23 CFR 650 Subpart A (650-A)

The state–of–practice assumes and applies riverine (or stillwater for coastal areas) conditions for the design frequency. FHWA is unaware of any State DOT that designs bridges for other than a peak flow.

The result is that in the coastal environment, with the assumption and focus of riverine peak flow, the bridge structures do not consider effects of wave action or other coastal hydraulic constituents. For example, a State DOT may apply the FEMA FIS, but only will use the storm surge (stillwater elevation) values and neglect the additional wave setup values. Additionally, even when they wish to consider such constituents (such as FDOT after Hurricane Ivan for the I-10 bridge over Escambia Bay), State DOTs find themselves in the position that their own regulations and guidelines do not permit them to consider alternative bridge design frequency criteria.

FHWA Regulatory Tools

From FHWA's perspective, 650-A provides the regulatory and policy framework to assist in this quandary. Specifically, there are several elements of 650-A that address or can apply to bridges in the coastal environment. They are mostly within section 650.115 (Design Standards):

[1] Typical practice is quite variable. Some State DOTs use the 50-year flood (Q_{50}), but require some associated freeboard between the water surface elevation and low chord. Others use the larger of 1) Q_{50} and freeboard or 2) the 100-year flood event. In some States, smaller bridges may use a 25-year return period as the design flow.

23 CFR 650.115 (Design Standards)

(a) The design selected for an encroachment shall be supported by analyses of design alternatives with consideration given to capital costs and risks, and to other economic, engineering, social and environmental concerns.

(1) Consideration of capital costs and risks shall include, as appropriate, a risk analysis or assessment which includes:

(i) The overtopping flood or the base flood, whichever is greater, or

(ii) The greatest flood which must flow through the highway drainage structure(s), where overtopping is not practicable. The greatest flood used in the analysis is subject to state-of-the-art capability to estimate the exceedance probability.

(2) The design flood for encroachments by through lanes of Interstate highways shall not be less than the flood with a 2 percent chance of being exceeded in any given year. No minimum design flood is specified for Interstate highway ramps and frontage roads or for other highways.

(3) Freeboard shall be provided, where practicable, to protect bridge structures from debris- and scour-related failure.

Applicability to Coastal Flood Frequencies

Several elements in the Design Standards have direct application for a hydraulic structure in a coastal environment in terms considering allowable design frequency (annual exceedance probability/return period) and coastal hydraulic constituents. Table 1 describes several of these elements, provides the associated section language, provides (if cited) the regulatory definition of the element, and some applicable discussion related to the coastal environment.

While Section 650.115(a)(2) specifies that the minimum design frequency for an Interstate is the 50-year flood (2-percent exceedance), the regulation allows consideration of larger design frequencies. Section 650.115 provides FHWA the latitude to allow a State DOT to design a bridge for at least the 100-year flood event. For the coastal environment, one of the most useful sections is 650.115(a)(1)(ii) and the concept of the "greatest flood."

The original intent of section 650.115(a)(1)(ii) described a bridge over a dam where no overtopping was possible. However, the section is equally applicable to a coastal bridge. The Design Standards allow a design to relate an historical storm event to the "greatest flood." Such an historic event may be associated with a frequency higher than the 100-year storm event.

However, the use of the "greatest flood" approach in a coastal environment also has the requirement to use "state-of-art" approaches to determine the associated probability of the event. Such a requirement prevents a designer from assigning an arbitrary high frequency to a project without regards to risk and cost. FHWA considers state-of-art to consist of having a scientifically and engineering robust methodology for determining the greatest flood. Qualified coastal engineers with experience in hydraulics and modeling must conduct this methodology.

Table 1. Focus on key portions and elements of the 650.115 Design Standards

Element	Definition (from 650.105)	650.115 Section(s)	Discussion
Risk	*"**Risk**" shall mean the consequences associated with the **probability of flooding** attributable to an encroachment. It shall include the potential for property loss and hazard to life during the service life of the highway.*	*… consideration given to capital costs and* **risks***…*	**Risk** (requires and) allows consideration of the **probability of flooding**, including higher return periods in design.
Risk analysis	*"**Risk analysis**" shall mean an economic comparison of design alternatives using expected total costs (construction costs plus risk costs) to determine the alternative with the least total expected cost to the public. It shall include probable flood-related costs during the service life of the facility for highway operation, maintenance, and repair, for highway-aggravated flood damage to other property, and for additional or interrupted highway travel.*	*… Consideration of capital costs and risks shall include, as appropriate, a* **risk analysis** *or assessment …*	**Risk analysis** implies looking at alternative design, including larger or more robust structures. The **risk analysis** includes consideration of least total economic costs (LTEC) of such a design. Flood damage costs are included in the risk analysis. For example, sizing a larger structure or increasing the elevation of the bridge low chord may be the most effective design when considering cost of successive in-place bridge replacement.
Overtopping Flood	*"**Overtopping flood**" shall mean the flood described by the **probability of exceedance** and water surface elevation at which flow occurs over the highway, over the watershed divide, or through structure(s) provided for emergency relief.*	*… The* **overtopping flood** *or the base flood, whichever is greater …*	This section relates the **overtopping flood** to some exceedance probability. The regulation does not necessarily just include bridges. For coastal roadway sections (e.g., roads along the Gulf Islands National Seashore), this overtopping flood element would allow consideration of higher design frequencies – even more than the 100-year base flood event.
Base Flood	*"**Base flood**" shall mean the flood or tide having a 1 percent chance of being exceeded in any given year.*	*… The overtopping flood or the* **base flood***, whichever is greater …*	The consideration of **base flood** allows the design to use the 100-year (1 percent annual exceedance) flood event. Typical practice assumes equivalence of the frequencies of storm events and flood events. Therefore, the 100-year rainfall may produce the 100-year flood event. While not as closely correlated, a 100-year wind field may produce an equivalent 100-year wave setup. When coupled to the overtopping flood, this section provides the ability to design for larger return periods than the typical 50-year event.
Greatest Flood	*Not defined in regulation*	*… The **greatest flood** which must flow through the highway drainage structure(s), where overtopping is not practicable ...*	The consideration of the **greatest flood** provides ability to consider additional storm events (including historical events) that are not covered by other Design Standard sections. In regards to major and critical bridges, overtopping is not a reasonable design consideration. Recent bridges failures modes (wave impact and wave uplift forces) demonstrate why overtopping is not an option at some coastal bridges.
State-of-art capability	*Not defined in regulation*	*… The greatest flood used in the analysis is subject to **state-of-the-art capability** to estimate the exceedance probability ...*	Applying **state-of-art** capability and approaches to describe the frequency of the event means having a robust scientific and engineering methodology and qualified coastal engineers with experience in hydraulics and modeling perform the analyses.

Table 1. Focus on key portions and elements of the 650.115 Design Standards

Element	Definition (from 650.105)	650.115 Section(s)	Discussion
Freeboard	*"Freeboard" shall mean the vertical clearance of the lowest structural member of the bridge superstructure above the water surface elevation of the overtopping flood.*	*… Freeboard shall be provided, where practicable, to protect bridge structures from debris- and scour-related failure.*	In the FHWA definition, no freeboard exists unless the low chord is above the overtopping flood elevation. Freeboard is a necessary consideration for any bridge structure. Given any uncertainty regarding the predicted water surface elevation, debris, or scour, prudent practice considers and adds some freeboard value to the low bridge chord. In a coastal environment, this uncertainty may include the effects of waves. The freeboard mitigates potential wave uplift and buoyancy impacts to the bridge substructure.

For example, after Hurricane Ivan and the resulting I-10 (Escambia Bay) failure, HIBT, after being consulted by the FHWA Florida Division and Florida Department of Transportation (FDOT), suggested using the greatest flood approach for the proposed new replacement design. From this recommendation, FDOT engaged the services of qualified coastal scientists and engineers who documented and determined the surge and wave heights associated with the Ivan event (about a 200-year frequency). FDOT was then able to determine the associated low chord height of the replacement bridge (and approaches).

Based on this approach, FHWA approved the hydraulic related elements of the replacement bridge. At that time, FHWA made it clear to FDOT that this use of the regulation (and use of a higher design frequency) was on a case-by-case basis. FHWA would not approve other bridge designs that deviated from FDOT design flood guidelines without similar, comprehensive state-of-art analyses.

Coastal Bridges and Freeboard

The FHWA regulatory definition (23 CFR 650 subpart A) describes that no freeboard exists unless low steel is above the overtopping flood elevation[2]. In other words, if low chord is not higher than the low point on the roadway, no freeboard exists. The use of the *overtopping flood elevation* in the definition associates freeboard with both uncertainty and frequencies higher than those normally used in design.

In a coastal environment, this uncertainty should include the effects of waves. The freeboard mitigates potential wave impact, uplift, and buoyancy forces on the bridge substructure. General freeboard considerations include:

- Determining the amount of freeboard should apply the same risk assessment and resulting design frequency associated with coastal bridge flooding analyses.
- Any analyses should consider values and characteristics of different wave categories (i.e., significant, two-percent, one-percent, and maximum wave heights, crest elevations and periods).

[2] This is a slightly different way of defining freeboard from usual [Corps/FEMA] interpretations.

Determining these characteristics for a design frequency provides insights such as:

- Applying current State DOT freeboard guidelines to the significant wave height may be appropriate in some cases, but not so in others.
- Because they do not occur as often as the significant wave, the maximum wave (or even the one- or two-percent waves) will not likely cause failure (or did not during Hurricanes Ivan and Katrina), so adding freeboard to avoid those waves may not be cost effective.
- Even so, knowing about wave types and characteristics does provide some sense of the adequacy of freeboard criteria. Otherwise, arbitrarily setting a freeboard may not ensure that a bridge is risk free.

For example, FDOT added freeboard to the I-10 bridge design. In doing so, FDOT considered the risks and costs of elevating the bridge low chord above the wave crest elevations of different waves (Ivan induced significant, one-percent, and maximum). FHWA approved this freeboard height using the same case-by-case criteria used in the wave height determination.

Coastal Bridges and Scour

The FHWA scour program requires that bridge owners evaluate bridges for potential scour associated with the 100-year (base flood) event and check for scour effects for the 500-year (superflood) event. For some coastal bridges, designers have discussed keeping bridges at the current low chord elevation, but adding structural measures (i.e., stronger connections between pier and decks) as a retrofit measure against wave impact and uplift forces. A FHWA concern is the possibility that such structural measures may also induce pressure scour conditions because as the bridge becomes overtopped, the vertical contraction of the bridge and streambed induce velocities and vortices conducive to scour formation. Therefore, such structural measures also need to evaluate the hydraulic and scour characteristics of such events. This evaluation will likely need to specify some form of scour countermeasure (e.g., deeper foundations, armoring, etc).

Approaches to Coastal Bridges

Generally, shallower waters or shoreline areas where 1) depth limited waves occur or 2) coastal flooding consists only of still water level (surge) will have reduced wave heights and therefore reduced wave loads on the structure. These areas are the typical locations of approaches to coastal bridges. Special attention should be given to the transition point between the low chord bridge elevation, the elevation and location of the touchdown, and the elevation of the remainder of the approach embankment.

Once again, FHWA regulations and programs provide a framework to balance cost and risk assessments following similar approaches and guidelines as before. Generally, the most critical portions are the 1) touchdown location and elevation and 2) design of transition height and length throughout the remainder of the approach embankment.

Bridge abutments are prone to scour and wave attack, and so these elements would be candidates for increased foundation depths, scour countermeasures, and possibly elevation. Analyses would include scour assessments from the 100-year and 500-year events.

As the approach extends further inshore, the less it will be subject to wave attack. Continuing to allow approach embankment roadway (or low chord) elevations similar to those found in open waters requires compelling results from coastal engineering hydraulic analyses. Otherwise, bridge approach design would be subject to normal geometric and structural specifications.

Summary

Coastal bridges exhibit different hydraulic conditions than riverine bridges. These conditions make such bridges more susceptible to flood damages during large storm events. Most State guidelines and regulations do not address designing a coastal bridge to allow higher flood frequencies. FHWA regulations do so, and can be applied on a case-by-case basis.

Not all coastal bridges need to be designed to these higher frequencies. FHWA should allow a State DOT to use federal-aid funds to design and construct coastal bridges to a higher frequency only after the State DOT obtains a state-of-art analysis of the coastal hydraulics and reviews associated frequencies. Most State DOTs (and FHWA) hydraulic units lack the expertise to conduct such analyses. A State DOT would need to retain the services of a qualified coastal engineer to conduct the state-of-art analysis to obtain the design frequency for coastal bridges. FHWA regulations require an assessment of capital costs and (most critically) risk in such events.

General Recommendations

The FHWA Office of Bridge Technology recommends considering the following flood frequency and freeboard guidance for the analysis and design of coastal bridges:

1. Use the typical State DOT design frequency (typically 50 year flood event and freeboard) for most coastal bridges. These bridges need to consider:
   - A combination of surge and wave effects (stillwater and wave crest).
   - Any likelihood of pressure scour during an overtopping episode.
   - Check the effect of the 100-year event significant wave crest elevation on the low chord (and add freeboard as needed).

2. Use a 100-year design frequency for Interstate, major structures, and critical bridges (i.e., serving as an emergency or evacuation route). These bridges need to consider:
   - A combination of surge and wave effects.
   - Any likelihood of pressure scour during any overtopping episode.
   - Check the effects of the overtopping storm frequency surge and waves on the low chord.
   - Such analyses subject to review by a qualified coastal engineer with experience in hydraulics, scour, and modeling.

3. Allow consideration of design frequencies higher than 100-year events:
    - On a case-by-case basis.
    - Only after the State DOT obtains a state-of-art analysis of the coastal hydraulics and associated frequencies.
    - Such analyses having been conducted by a qualified coastal engineer with experience in hydraulics, scour, and modeling.
    - Conducting risk and cost assessments.
    - If possible, determining the effects of the 500-year storm frequency surge and waves on the low chord.

4. Designing to super flood events (i.e., 500-year design frequency), would only be considered in extremely rare and compelling situations.

Specific Recommendations

For the critical bridges (portions of US 90 in Mississippi and I-10 in Louisiana) damaged by Hurricane Katrina, the FHWA Office of Bridge Technology recommends:

1. That they perform a series of coastal hydraulic analyses using the approach outlined in General Recommendation 3 (and as applied in the Hurricane Ivan/I-10 Escambia Bay analyses).

2. The findings from these analyses be included in any design build contract specifications.

3. The analyses consider Katrina as the "greatest flood" to establish frequency and coastal hydraulic values used in such a design.

For the FHWA Office of Bridge Technology

/s/
Joseph Krolak
FHWA Hydraulic Engineer

APPENDIX E

CHECKLIST AND GUIDELINES FOR REVIEW OF GEOTECHNICAL REPORTS AND PRELIMINARY PLANS AND SPECIFICATIONS

U.S. Department of Transportation
Federal Highway Administration

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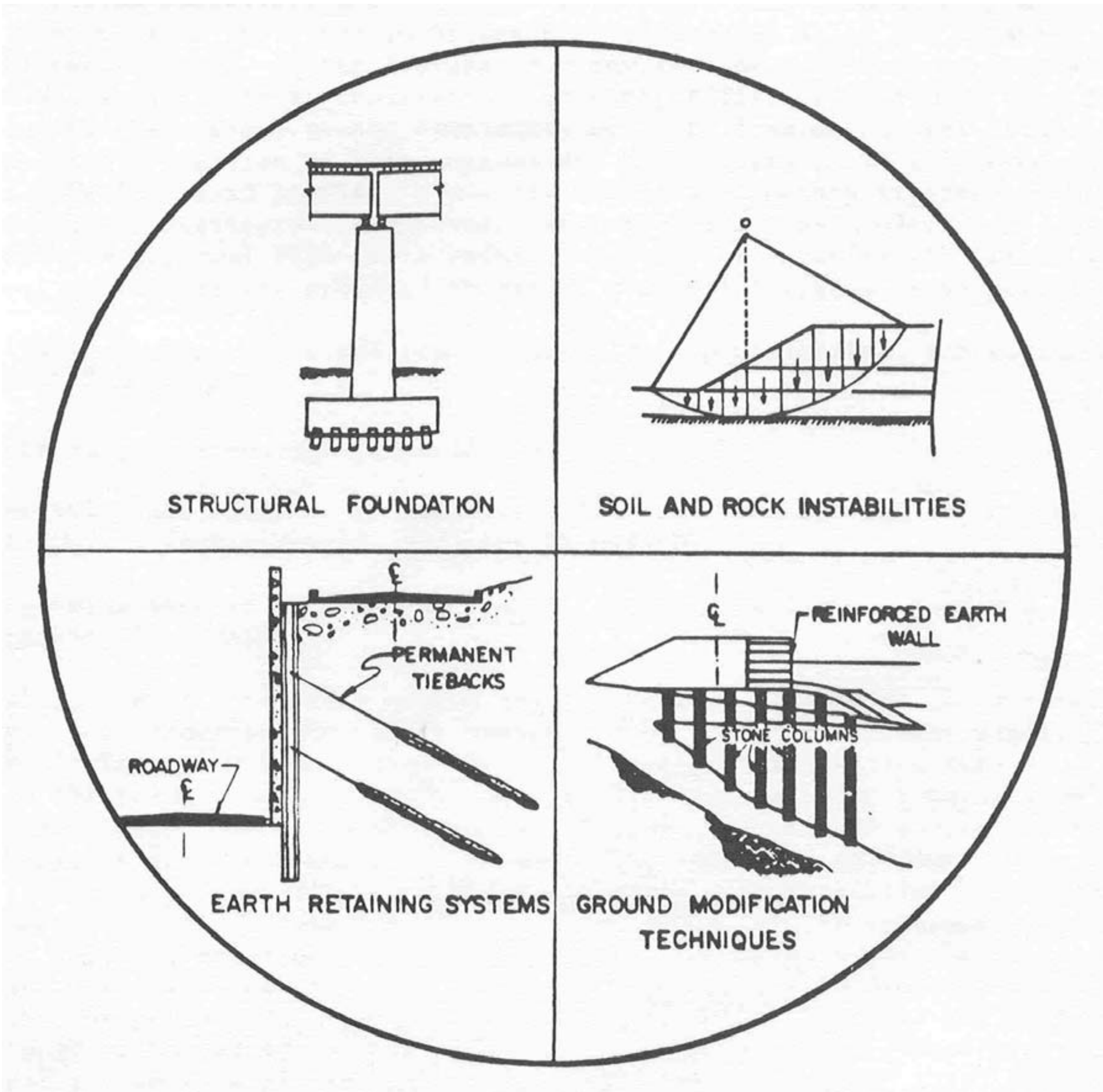

CHECKLIST AND GUIDELINES FOR REVIEW OF GEOTECHNICAL REPORTS AND PRELIMINARY PLANS AND SPECIFICATIONS

PREFACE

A set of review checklists and technical guidelines has been developed to aid engineers in their review of projects containing major and unusual geotechnical features. These features may involve any earthwork or foundation related activities such as construction of cuts, fills, or retaining structures, which due to their size, scope, complexity or cost, deserve special attention. A more specific definition of both unusual and major features is presented in Table 1. Table 1 also provides a description of a voluntary program by which FHWA generalists engineers determine what type and size projects may warrant a review by a FHWA geotechnical specialist. The review checklists and technical guidelines are provided to assist generalist highway engineers in:

- Reviewing both geotechnical reports and plan, specification, and estimate (PS&E)* packages;

- Recognizing cost-saving opportunities

- Identifying deficiencies or potential claim problems due to inadequate geotechnical investigation, analysis or design;

- Recognizing when to request additional technical assistance from a geotechnical specialist.

At first glance, the enclosed review checklists will seem to be inordinately lengthy, however, this should not cause great concern. First, approximately 50 percent of the review checklists deal with structural foundation topics, normally the primary responsibility of a bridge engineer; the remaining 50 percent deal with roadway design topics. Second, the general portion of the PS&E checklist is only one page in length. The remaining portions of the PS&E checklist apply to specific geotechnical features – such as pile foundations, embankments, landslide corrections, etc., and would only be completed when those specific features exist on the project. Third, the largest portion of the checklists deals with the review of geotechnical reports, with a separate checklist for each of eight geotechnical features. The checklist for each geotechnical feature is only one to two pages in length. Therefore, on most projects, reviewers will find that only a small portion of the total enclosed checklist needs to be completed.

* For purposes of this document, PS&E refers to a plan and specification review at any time during a project's development. Hence, the review may be at a preliminary or partial stage of plan development.

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GEOTECHNICAL REVIEW CHECKLISTS AND TECHNCIAL GUIDLINES

Introduction

The following review checklists and technical guidelines have been developed to aid engineers with review of geotechnical reports, plans and special provisions on projects containing major and unusual geotechnical features. These may involve any earthwork or foundation related activities such as construction of cuts, fills, or retaining structures, which due to their size, scope, complexity or cost, deserve special attention. A more specific definition of both major and unusual features is presented in Table 1. The checklists and review guidelines are intended to serve four primary purposes.

First, for projects that are submitted to a FHWA geotechnical specialist, the checklists and technical guidelines are provided to aid FHWA generalist engineers in making a quick review of the geotechnical report and accompanying support data provided by the State, to insure that the information provided by the State is complete enough to allow adequate technical review by the FHWA geotechnical specialist.

Second, for projects which will not be submitted to a FHWA geotechnical specialist for formal review (which will be the majority of projects handled by the FHWA division office) the checklists and technical guidelines are provided to assist generalist engineers in (1) reviewing geotechnical reports and preliminary plan and specification packages; (2) recognizing cost-saving opportunities; (3) spotting deficiencies or potential claim problems due to inadequate geotechnical investigations, analysis, or design; (4) recognizing when to request technical assistance for a FHWA geotechnical specialist.

Third, it should be noted that the checklists and technical guidelines also include coverage of structure foundations. These review checklists and technical guidelines have been developed to fill an existing need in this area.

Fourth, this document sets forth minimum geotechnical standards or criteria to show transportation agencies and consultants the basic geotechnical information which FHWA recommends be provided in geotechnical reports and PS&E packages.

TABLE 1
PROJECT REVIEW GUIDELINES

The following project review guidelines are given to assist FHWA generalist engineers in determining what type and size projects may warrant review by a FHWA geotechnical specialist.

A FHWA geotechnical specialist should review Geotechnical reports and supporting data for major or unusual geotechnical features, described below. The FHWA division office should also request FHWA geotechnical specialist review for any project that is considered to involve geotechnical risk or excessive expense in its design or construction. Supporting data for these reviews include preliminary plans, specifications, and cost estimates (if available at the time of geotechnical report submittal). Emphasis will be placed on review of these projects in the preliminary stage in order to optimize cost savings through early identification of potential problems or more innovative designs. To be of maximum benefit geotechnical reports and supporting data should be forwarded for review as soon as available, and at least 60 days prior to the scheduled project advertisement date. The review by the FHWA geotechnical specialist should be completed within 10 working days.

A. “Major” Geotechnical Features

Geotechnical reports and supporting data for major geotechnical project features should be submitted to the FHWA geotechnical specialist for review if the following project cost and complexity criteria exist:

	Cost Criteria
1. Earthwork – soil or rock cuts or fills where (a) the maximum height of cut or fill exceeds 15 m (50 ft), or (b) the cuts or fills are fills are located in topography and/or geological units with known stability problems.	Greater than $1,000,000
2. Soil and Rock Instability Corrections – cut, fill, or natural slopes which are presently or potentially unstable.	Greater than $ 500,000
3. Retaining Walls (geotechnical aspects) - maximum height at any point along the length exceeds 9 m (30 ft). Consideration of bidding cost-effective alternatives and geotechnical aspects (bearing capacity, settlement, overturning, sliding, etc.) are of prime concern. Structural design of and footings is beyond the scope of these reviews.	Greater than $ 250,000

B. “Unusual” Geotechnical Features

Geotechnical reports and supporting data for all projects containing unusual geotechnical features should be submitted to the FHWA geotechnical specialist for review.

An unusual geotechnical project feature is any geotechnical feature involving: (1) difficult or unusual problems, e.g. embankment construction on a weak and compressible foundation material (difficult) or fills constructed using degradable shale (unusual); (2) new or complex designs, e.g. geotextile soil reinforcement, permanent ground anchors, wick drains, ground improvement technologies; and (3) questionable design methods, e.g. experimental retaining wall systems, pile foundations where dense soils exists.

<u>What is a Geotechnical Report</u>?

The geotechnical report is the tool used to communicate the site conditions and design and construction recommendations to the roadway design, bridge design, and construction personnel. Site investigations for transportation projects have the objective of providing specific information on subsurface soil, rock, and water conditions. Interpretation of the site investigation information, by a geotechnical engineer, results in design and construction recommendations that should be presented in a project geotechnical report. <u>The importance of preparing an adequate geotechnical report cannot be overstressed</u>. The information contained in this report is referred to often during the design period, construction period, and frequently after completion of the project (resolving claims). Therefore, the report should be as clear, concise, and accurate. Both an adequate site investigation and a comprehensive geotechnical report are necessary to construct a safe, cost-effective project. Engineers need these reports to conduct an adequate review of geotechnical related features, e.g., earthwork and foundations.

The State or their consultant should prepare "Preliminary" geotechnical reports for submittal to the design team whenever this information will benefit the design process. Early submittal of geotechnical information and recommendations or engineering evaluation of preliminary data may be necessary to establish basic design concepts or design criteria. This is commonly the case on large projects or projects containing complex or difficult geotechnical problems where alignment and/or grade changes may be appropriate based on geotechnical recommendations. The development of a "Final" geotechnical report will not normally be completed until design has progressed to the point where specific recommendations can be made for all of the geotechnical aspects of the work. Final alignment, grade, and geometry will usually have been selected prior to issuance of the final geotechnical report.

While the geotechnical report content and format will vary by project size and highway agency, all geotechnical reports should contain certain <u>basic</u> essential information, including:

- Summary of all subsurface exploration data, including subsurface soil profile, exploration logs, laboratory or in situ test results, and ground water information;
- Interpretation and analysis of the subsurface data;
- Specific engineering recommendations for design;
- Discussion of conditions for solution of anticipated problems; and
- Recommended geotechnical special provisions.

It is suggested that the State routinely include this minimum information in the geotechnical report for Federal-Aid highway projects and that a copy of this report be supplied to the FHWA division office at the time when the report is internally distributed in the State.

For brevity in this document, the term geotechnical report will be used as a general term to cover all types of geotechnical reports, e.g., foundation report, centerline soils report, landslide study report, etc.

Use of Review Checklists and Technical Guidelines

Review checklists have been prepared for review of geotechnical reports and review of the geotechnical aspects of preliminary plans, specification and estimate (PS&E)* packages. To simplify their use, the checklists are set up in a question and answer format. The geotechnical report checklists (pages 11 through 27) cover the important information that should be presented in project geotechnical reports. The PS&E review checklists (pages 28 through 33) cover the geotechnical aspects, ranging from assuring continuity between the project geotechnical report and contract documents to avoiding common claim pitfalls. Items that are identified with an asterisk (*) are considered to be of major importance. A response other than (yes) or (N/A) for any of these checklist questions is cause to contact the appropriate geotechnical engineer for a clarification and/or to discuss the project.

Groups of related questions and, in some cases, individual questions have been cross referenced to the "Soils and Foundations Workshop Manual"** so as to provide the generalist engineer user a reference on basic geotechnical items. Technical guidelines are presented in Tables 1 through 4. Since it is not possible to establish strict criteria for all geotechnical information that should be obtained or geotechnical analysis that should be performed for a particular project, only general or minimum guidelines can be established. Table 1 provides definitions of both major and unusual features and guidelines as to which projects may be appropriate for review by the FHWA geotechnical specialist. Table 2 presents guideline minimum boring, sampling, and testing criteria for subsurface investigations that should be conducted for major or unusual geotechnical features. Table 3 presents general guidelines on the major types of geotechnical engineering analyses that are normally required for embankments and cut slopes, structure foundations, and retaining structures. Guidance is given for all major soil types. Table 4 presents a list of technical support data that should be provided for correction of soil and rock instabilities (landslides). Due to the unique situation that landslides present in terms of a major expenditure of funds for rehabilitation, a concise and specific list of necessary support information is warranted.

The enclosed review checklists and technical guidelines cover the following geotechnical features:

- Centerline Cuts and Embankments
- Embankments Over Soft Ground
- Landslide Corrections
- Retaining Structures
- Structure Foundations (spread footings, piles, drilled shafts)
- Ground Improvement Techniques
- Material Sites

*For the purposes of this document, PS&E refers to a plan and specification review at anytime during a project's development. Hence, the review may occur at a preliminary or partial stage of plan development.

** "Soils and Foundations Workshop Manual", Publication # FHWA NHI-00-045

Reviews made during the preliminary stage of project development will commonly consist of reviewing the geotechnical report only, since detailed plans and specifications may not yet be prepared.

When reviewing the PS&E, the plans, special provisions, and final geotechnical report should be examined together. A major aspect of the PS&E review of project geotechnical features is to verify that the major design and construction recommendations given in the geotechnical report have been properly incorporated into the plans and specifications. The practice of most highway agencies is to prepare a single geotechnical report that includes subsurface information, interpretations, and design and construction recommendations. However, some agencies prepare two separate reports; one report that only presents the factual subsurface data (made available to bidders), and a separate report or design memorandum (not made available to bidders) which contains the interpretation of subsurface conditions and the design and construction recommendations. These reports not only form the basis of technical reviews but should also be the agency's basis for design and construction of earthwork and foundation features.

The review checklists should be used as the working document while the guidelines in Tables 1 through 4, and the indicated sections of the "Soils and Foundations Workshop Manual" should be used as references. The checklist questions should be completed by referring to the geotechnical report and contract documents, the appropriate sections of the tables, and by use of engineering judgement. For each question, the reviewer should indicate a yes, no, or unknown or non-application response. Upon completion of the checklists, the reviewer should summarize the negative responses and discuss these with the appropriate geotechnical engineers to determine if additional follow-up is appropriate.

Seismic design of geotechnical features has not been considered in this document. For guidance the reader is referred to "Geotechnical Engineering Circular No. 3, Design Guidance: Geotechnical Earthquake Engineering for Highways, Volume I – Design Principles", FHWA SA-97-076. Seismic loads represent an extreme loading condition therefore relatively low factors of safety are generally considered acceptable in a pseudo-static analysis. Factors of safety on the order of 1.1 to 1.15 are typically used in practice for both bearing capacity and sliding resistance. The choice of the factor of safety and of the seismic coefficient are intimately linked. For instance, of a seismic coefficient equal to the PGA (divided by g) has been used in the pseudo-static analysis because the foundation cannot tolerate large movements, a factor of safety of 1.0 may be used. Alternatively, if the seismic coefficient is one-half the PGA and the soil is susceptible to a post-peak strength decrease, a factor of safety of 1.1 to 1.15 should be used.

TABLE 2

GUIDELINE “MINIMUM” BORING, SAMPLING, AND TESTING CRITERIA

The most important step in geotechnical design is to conduct an adequate subsurface investigation. The number, depth, spacing, and character of borings, sampling, and testing to be made in an individual exploration program are so dependent upon site conditions and the type of project and its requirements, that no “rigid” rules may be established. Usually the extent of work is established as the site investigation progresses in the field. However, the following are considered reasonable “guidelines” to follow to produce the <u>minimum</u> subsurface data needed to allow cost-effective geotechnical design and construction and to minimize claim problems. (Reference: “Subsurface Investigations” FHWA HI-97-021)

Geotechnical Feature	Minimum Number of Borings	Minimum Depth of Borings
Structure Foundation	1 per substructure unit under 30 m (100 ft) in width 2 per substructure unit over 30 m (100 ft) in width Additional borings in areas of erratic subsurface conditions	Spread footings: 2B where L< 2B, 4B where L > 2B and interpolate for L between 2B and 4B Deep foundations: 6m (20ft) below tip elevation or two times maximum pile group dimension, whichever is greater If bedrock is encountered: for piles core 3 m (10 ft) below tip elevation; for shafts core 3D or 2 times maximum shaft group dimension below tip elevation, whichever is greater.
Retaining Structures	Borings spaced every 30 to 60 m (100 to 200 ft). Some borings should be at the front of and some in back of the wall face.	Extend borings to depth of 0.75 to 1.5 times wall height When stratum indicates potential deep stability or settlement problem, extend borings to hard stratum
Bridge Approach Embankments over Soft Ground	When approach embankments are to be placed over soft ground, at least one boring should be made at each embankment to determine the problems associated with stability and settlement of the embankment. Typically, test borings taken for the approach embankments are located at the proposed abutment locations to serve a dual function.	Extend borings into competent material and to a depth where added stresses due to embankment load is less than 10% of existing effective overburden stress or 3 m (10 ft) into bedrock if encountered at a shallower depth Additional shallow explorations (hand auger holes) taken at approach embankment locations to determine depth and extent of unsuitable surface soils or topsoil.
Centerline Cuts and Embankments	Borings typically spaced every 60 m (200 ft) (erratic conditions) to 120 m (400 ft) (uniform conditions) with at least one boring taken in each separate landform. For high cuts and fills, should have a minimum of 3 borings along a line perpendicular to centerline or planned slope face to establish geologic cross-section for analysis.	Cuts: (1) in stable materials extend borings minimum 5 m (15 ft) below depth of cut at the ditch line and, (2) in weak soils extend borings below grade to firm materials or to twice the depth of cut whichever occurs first. Embankments: Extend borings to a hard stratum or to a depth of twice the embankment height.
Landslides	Minimum 3 borings along a line perpendicular to centerline or planned slope face to establish geologic cross-section for analysis. Number of sections depends on extent of stability problem. For active slide, place at least on boring each above and below sliding area	Extend borings to an elevation below active or potential failure surface and into hard stratum, or to a depth for which failure is unlikely because of geometry of cross-section. Slope inclinometers used to locate the depth of an active slide must extend below base of slide.
Ground Improvement Techniques	Varies widely depending in the ground improvement technique(s) being employed. For more information see “Ground Improvement Technical Summaries” FHWA SA-98-086R.	
Material Sites (Borrow sources, Quarries)	Borings spaced every 30 to 60 m (100 to 200 ft).	Extend exploration to base of deposit or to depth required to provide needed quantity.

TABLE 2 (Continued)

GUIDELINE “MINIMUM” BORING, SAMPLING, AND TESTING CRITERIA

Sand or Gravel Soils SPT (split-spoon) samples should be taken at 1.5 m (5 ft) intervals or at significant changes in soil strata. Continuous SPT samples are recommended in the top 4.5 m (15 ft) of borings made at locations where spread footings may be placed in natural soils. SPT jar or bag samples should be sent to lab for classification testing and verification of field visual soil identification.
Silt or Clay Soils SPT and “undisturbed” thin wall tube samples should be taken at 1.5 m (5 ft) intervals or at significant changes in strata. Take alternate SPT and tube samples in same boring or take tube samples in separate undisturbed boring. Tube samples should be sent to lab to allow consolidation testing (for settlement analysis) and strength testing (for slope stability and foundation bearing capacity Analysis). Field vane shear testing is also recommended to obtain in-place shear strength of soft clays, silts and well-rotted peat.
Rock Continuous cores should be obtained in rock or shales using double or triple tube core barrels. In structural foundation investigations, core a minimum of 3 m (10 ft) into rock to insure it is bedrock and not a boulder. Core samples should be sent to the lab for possible strength testing (unconfined compression) if for foundation investigation. Percent core recovery and RQD value should be determined in field or lab for each core run and recorded on boring log.
Groundwater Water level encountered during drilling, at completion of boring, and at 24 hours after completion of boring should be recorded on boring log. In low permeability soils such as silts and clays, a false indication of the water level may be obtained when water is used for drilling fluid and adequate time is not permitted after boring completion for the water level to stabilize (more than one week may be required). In such soils a plastic pipe water observation well should be installed to allow monitoring of the water level over a period of time. Seasonal fluctuations of water table should be determined where fluctuation will have significant impact on design or construction (e.g., borrow source, footing excavation, excavation at toe of landslide, etc.). Artesian pressure and seepage zones, if encountered, should also be noted on the boring log. In landslide investigations, slope inclinometer casings can also serve as water observations wells by using “leaky” couplings (either normal aluminum couplings or PVC couplings with small holes drilled through them) and pea gravel backfill. The top 0.3 m (1 ft) or so of the annular space between water observation well pipes and borehole wall should be backfilled with grout, bentonite, or sand-cement mixture to prevent surface water inflow which can cause erroneous groundwater level readings.
Soil Borrow Sources Exploration equipment that will allow direct observation and sampling of the subsurface soil layers is most desirable for material site investigations. Such equipment that can consist of backhoes, dozers, or large diameter augers, is preferred for exploration above the water table. Below the water table, SPT borings can be used. SPT samples should be taken at 1.5 m (5 ft) intervals or at significant changes in strata. Samples should be sent to lab for classification testing to verify field visual identification. Groundwater level should be recorded. Observations wells should be installed to monitor water levels where significant seasonal fluctuation is anticipated.
Quarry Sites Rock coring should be used to explore new quarry sites. Use of double or triple tube core barrels is recommended to maximize core recovery. For riprap source, spacing of fractures should be carefully measured to allow assessment of rock sizes that can be produced by blasting. For aggregate source, the amount and type of joint infilling should be carefully noted. If assessment is made on the basis of an existing quarry site face, it may be necessary to core or use geophysical techniques to verify that nature of rock does not change behind the face or at depth. Core samples should be sent to lab for quality tests to determine suitability for riprap or aggregate.

TABLE 3

REQUIRED GEOTECHNICAL ENGINEERING ANALYSIS

Soil Classification			Embankment and Cut Slopes		Structure Foundations (Bridges and Retaining Structures)		Retaining Structures (Conventional, Crib and MSE)	
Unified	AASHTO[1]	Soil Type	Slope Stability[2] Analysis	Settlement Analysis	Bearing Capacity Analysis	Settlement Analysis	Lateral Earth Pressure	Stability Analysis
GW GP GM GC SW SP SM SC	A-1-a A-1-a A-1-b A-2-6 A-2-7 A-1-b A-3 A-2-4 A-2-5 A-2-6 A-2-7	GRAVEL Well-graded GRAVEL Poorly-graded GRAVEL Silty GRAVEL Clayey SAND Well-graded SAND Poorly-graded SAND Silty SAND Clayey	Generally not required if cut or fill slope is 1.5H to 1V or flatter, and underdrains are used to draw down the water table in a cut slope. Erosion of slopes may be a problem for SW or SM soils.	Generally not required except possibly for SC soils.	Required for spread footings, pile or drilled shaft foundations. Spread footings generally adequate except possibly for SC soils	Generally not needed except for SC soils or for large, heavy structures. Empirical correlations with SPT values usually used to estimate settlement	GW, SP, SW & SP soils generally suitable for backfill behind or in retaining or reinforced soil walls. GM, GC, SM & SC soils generally suitable if have less than 15% fines. Lateral earth pressure analysis required using soil angle of internal friction.	All walls should be designed to provide minimum F.S. = 2 against overturning & F.S. = 1.5 against sliding along base. External slope stability considerations same as previously given for cut slopes & embankments.
ML	A-4	SILT Inorganic silt Sandy	Required unless non-plastic. Erosion of slopes may be a problem.	Required unless non-plastic.	Required. Spread footing generally adequate.	Required. Can use SPT values if non-plastic.	These soils are not recommended for use directly behind or in retaining or reinforced soil walls.	
CL	A-6	CLAY Inorganic Lean Clay	Required	Required				
OL	A-4	SILT Organic	Required	Required				

[1] This is an approximate correlation to Unified (Unified Soil Classification system is preferred for geotechnical engineering usage, AASHTO system was developed for rating pavement subgrades).

[2] These are general guidelines, detailed slope stability analysis may not be required where past experience in area is similar or rock gives required slope angles.

TABLE 3 (Continued)

Soil Classification			Embankment and Cut Slopes		Structure Foundations (Bridges and Retaining Structures)		Retaining Structures (Conventional, Crib and MSE)	
Unified	AASHTO[1]	Soil Type	Slope Stability[2] Analysis	Settlement Analysis	Bearing Capacity Analysis	Settlement Analysis	Lateral Earth Pressure	Stability Analysis
MH	A-5	SILT Inorganic	Required. Erosion of slopes may be a problem.	Required.	Required. Deep foundation generally required unless soil has been preloaded.	Required. Consolidation test data needed to estimate settlement amount and time.	These soils are not recommended for use directly behind or in retaining walls.	All walls should be designed to provide minimum F.S. = 2 against overturning & F.S. = 1.5 against sliding along base. External slope stability considerations same as previously given for cut slopes & embankments
CH	A-7	CLAY Inorganic Fat Clay	Required.	Required.				
OH	A-7	CLAY Organic	Required.	Required.				
PT	----	PEAT Muck	Required.	Required. Long term settlement can be significant	Deep foundation required unless peat excavated and replaced.	Highly compressible and not suitable for foundation support		
Rock			Fills – not required for slopes 1.5H to 1V or flatter. Cuts – required but depends on spacing, orientation and strength of discontinuities and durability of rock		Required for spread footings or drilled shafts. Empirically related to RQD[3]	Required where rock is badly weathered or closely fractured (low RQD). May require in situ test such as pressuremeter.	Required. Use rock backfill angle of internal friction.	

REMARKS:

Soils – temporary ground water control may be needed for foundation excavations in GW through SM soils.

Backfill specifications for reinforced soil walls using metal reinforcements should meet the following requirements in insure use of non-corrosive backfill:

pH range = 5 to 10; Resistivity > 3000 ohm-cm; Chlorides < 100 ppm; Sulfates < 200 ppm; Organic content 1% maximum

Rock – Durability of shales (siltstone, claystone, mudstone, etc.) to be used in fills should be checked. Non-durable shales should be embanked as soils, i.e., placed in maximum 0.3 m (1 ft) loose lifts and compacted with heavy sheepsfoot or grid rollers.

[1] This is an approximate correlation to Unified (Unified Soil Classification system is preferred for geotechnical engineering usage, AASHTO system was developed for rating pavement subgrades).

[2] These are general guidelines, detailed slope stability analysis may not be required where past experience in area is similar or rock gives required slope angles.

[3] RQD (Rock Quality Designation) = sum of pieces of rock core 4” or greater in length divided by the total length of core run.

TABLE 4
CORRECTION OF SOIL AND ROCK-RELATED INSTABLIITIES

Each year hundreds of millions of dollars are spent to correct soil or rock-related instabilities on highways. The purpose of this technical note is to advise field engineers what technical support information is essential such that a complete evaluation can be performed. For the purpose of this technical note, soil and rock-related instabilities are defined as follows: “A condition that currently or threatens to affect the stability or performance the stability or performance of a highway facility and is the result of the inadequate performance of the soil or rock components.” This includes major instabilities resulting form or associated with: landslides, rockfalls, sinkholes, and degrading shales. Technical support data needed are:

1. Site plan and typical cross-section(s) representing ground surface conditions prior to failure, along with subsurface configuration after failure. Photographs, including aerials, if available, would also be beneficial.

2. Cross-section(s) showing soil and/or rock conditions and water bearing strata as determined by drilling and possibly geophysical surveys.

3. Description of the latent state of the unstable mass, whether movement has stopped or is still occurring, and if so, at what rate.

4. Boring logs.

5. Instrumentation data and/or other information used to define the depth and location of the failure zone. The underground location of the failure zone should be shown on the cross-section(s).

6. Shear strength test data and a description of the testing method utilized on the materials, through which failure is occurring. Where average shear strength is calculated using an assumed failure surface and a factor of safety of 1.0, the complete analysis should be provided and location of assumed water table(s) shown.

7. Proposed corrective schemes including: estimated costs, final safety factors, and design analysis for each alternative solution.

8. Narrative report containing instability history; record of maintenance costs and activity, and preventative measures taken, if any; reasons for inadequacy of the original design; description and results of subsurface investigation performed; summary and results of stability analysis performed; and recommendations for correction.

GEOTECHNICAL REPORT REVIEW CHECKLISTS

The following checklists cover the major information and recommendations that should be addressed in project geotechnical reports.

Section A covers site investigation information that will be common to all geotechnical reports for any type of geotechnical feature.

Sections B through I cover the basic information and recommendations that should be presented in geotechnical reports for specific geotechnical features: centerline cuts and embankments, embankments over soft ground, landslides, retaining structures, structure foundations and material sites.

Subject Page

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In most sections and subsections the user has been provided supplemental page references to the "Soils and Foundations Workshop Manual" FHWA NHI-00-045. These page numbers appear in parentheses ( ) immediately adjacent to the section or subsection topic. Generalist engineers are particularly encouraged to read these references. Additional reference information on these topics is available in the Geotechnical Engineering Notebook, a copy of which is kept in all FHWA Division offices by either the Bridge Engineer or the engineer with the geotechnical collateral duty.

Certain checklist items are of vital importance to have been included in the geotechnical report. These checklist items have been marked with an asterisk (*). A negative response to any of these asterisked items is cause to contact the geotechnical engineer for clarification of this omission.

GTR REVIEW CHECKLIST FOR SITE INVESTIGATION

A. Site Investigation Information

Since the most important step in the geotechnical design process is to conduct an adequate site investigation, presentation of the subsurface information in the geotechnical report and on the plans deserves careful attention.

Geotechnical Report Text (Introduction) (Pgs. 10-1 to 10-4)	Yes	No	Unknown or N/A
1. Is the general location of the investigation described and/or a vicinity map included?	___	___	_____
2. Is scope and purpose of the investigation summarized?	___	___	_____
3. Is concise description given of geologic setting and topography of area?	___	___	_____
4. Are the field explorations and laboratory tests on which the report is based listed?	___	___	_____
5. Is the general description of subsurface soil, rock, and groundwater conditions given?	___	___	_____
*6. Is the following information included with the geotechnical report (typically included in the report appendices):			
a. Test hole logs? (Pgs. 2-24 to 2-32)	___	___	_____
b. Field test data?	___	___	_____
c. Laboratory test data? (Pgs. 4-22 to 4-23)	___	___	_____
d. Photographs (if pertinent)?	___	___	_____
Plan and Subsurface Profile (Pgs. 2-19, 3-9 to 3-12, 10-13)			
*7. Is a plan and subsurface profile of the investigation site provided?	___	___	_____
8. Are the field explorations located on the plan view?	___	___	_____

*A response other than (yes) or (N/A) for any of these checklist questions is cause to contact the appropriate geotechnical engineer for a clarification and/or to discuss the project.

		Yes	No	Unknown or N/A
A.	Site Investigation Information (Cont.)			
*9.	Does the conducted site investigation meet minimum criteria outlined in Table 2?	___	___	_____
10.	Are the explorations plotted and correctly numbered on the profile at their true elevation and location?	___	___	_____
11.	Does the subsurface profile contain a word description and/or graphic depiction of soil and rock types?	___	___	_____
12.	Are groundwater levels and date measured shown on the subsurface profile?	___	___	_____
	Subsurface Profile or Field Boring Log (Pgs. 2-14, 2-15, 2-24 to 2-31)			
13.	Are sample types and depths recorded?	___	___	_____
*14.	Are SPT blow count, percent core recovery, and RQD values shown?	___	___	_____
15.	If cone penetration tests were made, are plots of cone resistance and friction ratio shown with depth?			
	Laboratory Test Data (Pgs. 4-6, 4-22, 4-23)			
*16.	Were lab soil classification tests such as natural moisture content, gradation, Atterberg limits, performed on selected representative samples to verify field visual soil identification?	___	___	_____
17.	Are laboratory test results such as shear strength (Pg. 4-14), consolidation (Pg. 4-9), etc., included and/or summarized?	___	___	_____

*A response other than (yes) or (N/A) for any of these checklist questions is cause to contact the appropriate geotechnical engineer for a clarification and/or to discuss the project.

GTR REVIEW CHECKLIST FOR CENTERLINE CUTS AND EMBANKMENTS

B. Centerline Cuts and Embankments (Pgs. 2-2 to 2-6)

In addition to the basic information listed in Section A, is the following information provided in the project geotechnical report.

Are station-to-station descriptions included for:	Yes	No	Unknown or N/A
1. Existing surface and subsurface drainage?	___	___	_____
2. Evidence of springs and excessively wet areas?	___	___	_____
3. Slides, slumps, and faults noted along the alignment?	___	___	_____

Are station-to-station recommendations included for the following?

General Soil Cut or Fill

4. Specific surface/subsurface drainage recommendations?	___	___	_____
5. Excavation limits of unsuitable materials?	___	___	_____
*6. Erosion protection measures for back slopes, side slopes, and ditches, including riprap recommendations or special slope treatment.	___	___	_____

Soil Cuts (Pgs. 5-23, 5-24)

*7. Recommended cut slope design?	___	___	_____
8. Are clay cut slopes designed for minimum F.S. = 1.50?	___	___	_____
9. Special usage of excavated soils?	___	___	_____
10. Estimated shrink-swell factors for excavated materials?	___	___	_____
11. If answer to 3 is yes, are recommendations provided for design treatment?	___	___	_____

*A response other than (yes) or (N/A) for any of these checklist questions is cause to contact the appropriate geotechnical engineer for a clarification and/or to discuss the project.

B.	Centerline Cuts and Embankments (Cont.)	Yes	No	Unknown or N/A
	Fills (Pgs. 5-1 to 5-3)			
12.	Recommended fill slope design?	___	___	_____
13.	Will fill slope design provide minimum F.S. = 1.25?	___	___	_____
	Rock Slopes			
*14.	Are recommended slope designs and blasting specifications provided?	___	___	_____
*15.	Is the need for special rock slope stabilization measures, e.g., rockfall catch ditch, wire mesh slope protection, shotcrete, rock bolts, addressed?	___	___	_____
16.	Has the use of "template" designs been avoided (such as designing all rock slopes on 0.25:1 rather than designing based on orientation of major rock jointing)?	___	___	_____
*17.	Have effects of blast induced vibrations on adjacent structures been evaluated?	___	___	_____

*A response other than (yes) or (N/A) for any of these checklist questions is cause to contact the appropriate geotechnical engineer for a clarification and/or to discuss the project.

GTR REVIEW CHECKLIST FOR EMBANKMENTS OVER SOFT GROUND

C. Embankments Over Soft Ground

Where embankments must be built over soft ground (such as soft clays, organic silts, or peat), stability and settlement of the fill should be carefully evaluated. In addition to the basic information listed in Section A, is the following information provided in the project geotechnical report?

Embankment Stability (Pgs. 5-1 to 5-3, 5-20 to 5-22)	Yes	No	Unknown or N/A
*1. Has the stability of the embankment been evaluated for minimum F.S. = 1.25 for side slope and 1.30 for end slope of bridge approach embankments?	___	___	_____
*2. Has the shear strength of the foundation soil been determined from lab testing and/or field vane shear or cone penetrometer tests?	___	___	_____
*3. If the proposed embankment does not provide minimum factors of safety given above, are recommendations given or feasible treatment alternates, which will increase factor of safety to minimum acceptable (such as change alignment, lower grade, use stabilizing counterberms, excavate and replace weak subsoil, lightweight fill, geotextile fabric reinforcement, etc.)?	___	___	_____
*4. Are cost comparisons of treatment alternates given and a specific alternate recommended?	___	___	_____
Settlement of Subsoil (Pgs. 6-7 to 6-20)			
5. Have consolidation properties of fine-grained soils been determined from laboratory consolidation tests?	___	___	_____
*6. Have settlement amount and time been estimated?	___	___	_____
7. For bridge approach embankments, are recommendations made to get the settlement out before the bridge abutment is constructed (waiting period, surcharge, or wick drains)?	___	___	_____

*A response other than (yes) or (N/A) for any of these checklist questions is cause to contact the appropriate geotechnical engineer for a clarification and/or to discuss the project.

C. Embankments Over Soft Ground (Cont.)	Yes	No	Unknown or N/A
8. If geotechnical instrumentation is proposed to monitor fill stability and settlement, are detailed recommendations provided on the number, type, and specific locations of the proposed instruments?	___	___	_____
Construction Considerations (Pgs. 10-8, 10-9)			
9. If excavation and replacement of unsuitable shallow surface deposits (peat, muck, top soil) is recommended, are vertical and lateral limits of recommended excavation provided?	___	___	_____
10. Where a surcharge treatment is recommended, are plan and cross-section of surcharge treatment provided in geotechnical report for benefit of the roadway designer?	___	___	_____
11. Are instructions or specifications provided concerning instrumentation, fill placement rates and estimated delay times for the contractor?	___	___	_____
12. Are recommendations provided for disposal of surcharge material after the settlement period is complete?	___	___	_____

*A response other than (yes) or (N/A) for any of these checklist questions is cause to contact the appropriate geotechnical engineer for a clarification and/or to discuss the project.

GTR REVIEW CHECKLIST FOR LANDSLIDE CORRECTIONS

D. Landslide Corrections (Pgs. 5-1 to 5-4, 5-17 to 5-20)

In addition to the basic information listed in Section A, is the following information provided in the landslide study geotechnical report? (Refer to Table 4 for guidance on the necessary technical support data for correction of slope instabilities.)

		Yes	No	Unknown or N/A
*1.	Is a site plan and scaled cross-section provided showing ground surface conditions both before and after failure?	___	___	_____
*2.	Is the past history of the slide area summarized, including movement history, summary of maintenance work and costs, and previous corrective measures taken, if any?	___	___	_____
*3.	Is a summary given of results of site investigation, field and lab testing, and stability analysis, including cause(s) of the slide?	___	___	_____
	Plan			
4.	Are detailed slide features, including location of ground surface cracks, head scarp, and toe bulge, shown on the site plan?	___	___	_____
	Cross-section			
*5.	Are the cross-sections used for stability analysis included with the soil profile, water table, soil unit weights, soil shear strengths, and failure plane shown as it exists?	___	___	_____
6.	Is slide failure plane location determined from slope indicators?	___	___	_____
*7.	For an active slide, was soil strength along the slide failure plane back-calculated using a F.S. = 1.0 at the time of failure?	___	___	_____

*A response other than (yes) or (N/A) for any of these checklist questions is cause to contact the appropriate geotechnical engineer for a clarification and/or to discuss the project.

D.	Landslide Corrections (Cont.)	Yes	No	Unknown or N/A
	Text			
*8.	Is the following information presented for each proposed correction alternative (typical correction methods include buttress, shear key, rebuild slope, surface drainage, subsurface drainage-interceptor, drain trenches or horizontal drains, etc.).			
a.	Cross-section of proposed alternative?	___	___	_____
b.	Estimated safety factor?	___	___	_____
c.	Estimated cost?	___	___	_____
c.	Advantages and disadvantages?	___	___	_____
9.	Is recommended correction alternative(s) given that provide a minimum F.S. = 1.25?	___	___	_____
10.	If horizontal drains are proposed as part of slide correction, has subsurface investigation located definite water bearing strata that can be tapped with horizontal drains?	___	___	_____
11.	If a toe counterberm is proposed to stabilize an active slide has field investigation confirmed that the toe of the existing slide does not extend beyond the toe of the proposed counterberm?	___	___	_____
	Construction considerations			
12.	Where proposed correction will require excavation into the toe of an active slide (such as for buttress or shear key) has the “during construction backslope F.S.” with open excavation been determined?	___	___	_____
13.	If open excavation F.S. is near 1.0, has excavation stage stage construction been proposed?	___	___	_____
14.	Has seasonal fluctuations of groundwater table been considered?	___	___	_____
15.	Is stability of excavation backslope to be monitored?	___	___	_____
16.	Are special construction features, techniques and materials described and specified?	___	___	_____

*A response other than (yes) or (N/A) for any of these checklist questions is cause to contact the appropriate geotechnical engineer for a clarification and/or to discuss the project.

GTR REVIEW CHECKLIST FOR RETAINING STRUCTURES

E. Retaining Structures (See “Earth Retaining Structures” FHWA NHI-99-025)

In addition to the basic information listed in Section A, is the following information provided in the project geotechnical report?

		Yes	No	Unknown or N/A
*1.	Recommended soil strength parameters and groundwater elevations for use in computing wall design lateral earth pressures and factor of safety for overturning, sliding, and external slope stability.	___	___	_____
2.	Is it proposed to bid alternate wall designs?	___	___	_____
*3.	Are acceptable reasons given for the choice and/or exclusion of certain wall types?	___	___	_____
*4.	Is an analysis of the wall stability included with minimum acceptable factors of safety against overturning (F.S. = 2.0), sliding (F.S. = 1.5), and external slope stability (F.S. = 1.5)?	___	___	_____
5.	If wall will be placed on compressible foundation soils, is estimated total, differential and time rate of settlement given?	___	___	_____
6.	Will wall types selected for compressible foundation soils allow differential movement without distress?	___	___	_____
7.	Are wall drainage details, including materials and compaction, provided?	___	___	_____
	Construction Considerations			
8.	Are excavation requirements covered including safe slopes for open excavations or need for sheeting or shoring?	___	___	_____
9.	Fluctuation of groundwater table?	___	___	_____

*A response other than (yes) or (N/A) for any of these checklist questions is cause to contact the appropriate geotechnical engineer for a clarification and/or to discuss the project.

Top-down Construction Type Walls (See "Manual for Design & Construction Monitoring of Soil Nail Walls", FHWA SA-96-069R and "Ground Anchors and Anchored Systems", FHWA IF-99-015)

		Yes	No	Unknown or N/A
*10.	For soil nail and anchor walls are the following included in the geotechnical report?			
	a. Design soil parameters (ϕ, c, γ)	___	___	_____
	b. Minimum bore size (soil nails)?	___	___	_____
	c. Design pullout resistance (soil nails)?	___	___	_____
	d. Ultimate anchor capacity (anchors)?	___	___	_____
	e. Corrosion protection requirements?	___	___	_____

*A response other than (yes) or (N/A) for any of these checklist questions is cause to contact the appropriate geotechnical engineer for a clarification and/or to discuss the project.

GTR REVIEW CHECKLIST FOR SPREAD FOOTINGS

F. Structure Foundations – Spread Footings (Pgs. 7-1 to 7-17)

In addition to the basic information listed in Section A, is the following information provided in the project foundation report?

		Yes	No	Unknown or N/A
*1.	Are spread footing recommended for foundation support? If not, are reasons for not using them discussed?	___	___	_____
	If spread footing supports are recommended, are conclusions and recommendations given for the following:			
*2.	Is recommended bottom of footing elevation and reason for recommendation (e.g., based on frost depth, estimated scour depth, or depth to competent bearing material) given?	___	___	_____
*3.	Is recommended allowable soil or rock bearing pressure given?	___	___	_____
*4.	Is estimated footing settlement and time given?	___	___	_____
*5.	Where spread footings are recommended to support abutments placed in the bridge end fill, are special gradation and compaction requirements provided for select end fill and backwall drainage material (Pgs. 6-1 to 6-4)	___	___	_____
Construction Considerations				
6.	Have the materials been adequately described on which the footing is to be placed so the project inspector can verify that material is as expected?	___	___	_____
7.	Have excavation requirements been included for safe slopes in open excavations, need for sheeting or shoring, etc.?	___	___	_____
8.	Has fluctuation of the groundwater table been addressed?	___	___	_____

*A response other than (yes) or (N/A) for any of these checklist questions is cause to contact the appropriate geotechnical engineer for a clarification and/or to discuss the project.

GTR REVIEW CHECKLIST FOR DRIVEN PILES

G. Structure Foundations – Driven Piles (Pgs. 8-1 to 8-29, 9-1 to 9-35)

In addition to the basic information listed in Section A, if pile support is recommended or given as an alternative, conclusions/recommendations should be provided in the project geotechnical report for the following:

		Yes	No	Unknown or N/A
*1.	Is the recommended pile type given (displacement, non-displacement, steel pipe, concrete, H-pile, etc.) with valid reasons given for choice and/or exclusion? (Pgs. 8-1 to 8-3)	___	___	_____
2.	Do you consider the recommended pile type(s) to be the most suitable and economical?	___	___	_____
*3.	Are estimated pile lengths and estimated tip elevations given for the recommended allowable pile design loads?	___	___	_____
4.	Do you consider the recommended design loads to be reasonable?	___	___	_____
5.	Has pile group settlement been estimated (only of practical significance for friction pile groups ending in cohesive soil)? (Pgs. 8-20 to 8-22)	___	___	_____
6.	If a specified or minimum pile tip elevation is recommended, is a clear reason given for the required tip elevation, such as underlying soft layers, scour, downdrag, piles uneconomically long, etc.?	___	___	_____
*7.	Has design analysis (wave equation analysis) verified that the recommended pile section can be driven to the estimated or specified tip elevation without damage (especially applicable where dense gravel-cobble-boulder layers or other obstructions have to be penetrated)?	___	___	_____
8.	Where scour piles are required, have pile design and driving criteria been established based on mobilizing the full pile design capacity below the scour zone?	___	___	_____

*A response other than (yes) or (N/A) for any of these checklist questions is cause to contact the appropriate geotechnical engineer for a clarification and/or to discuss the project.

G.	Structure Foundations – Driven Piles (Cont.)	Yes	No	Unknown or N/A
9.	Where lateral load capacity of large diameter piles is an important design consideration, are p-y curves (load vs. deflection) or soil parameters given in the geotechnical report to allow the structural engineer to evaluate lateral load capacity of all piles?	___	___	_____
*10.	For pile supported bridge abutments over soft ground:			
a.	Has abutment downdrag load been estimated and solutions such bitumen coating been considered in design? Not generally required if surcharging of the fill is being performed. (Pgs. 8-21, 8-23)	___	___	_____
b.	Is bridge approach slab recommended to moderate differential settlement between bridge ends and fill?	___	___	_____
c.	If the majority of subsoil settlement will not be removed prior to abutment construction (by surcharging), has estimate been made of abutment rotation that can occur due to lateral squeeze of soil subsoil? (Pgs. 5-25, 5-26)	___	___	_____
d.	Does the geotechnical report specifically alert the structural designer to the estimated horizontal abutment movement?	___	___	_____
11.	If bridge project is large, has pile load test program been recommended? (Pgs. 9-23 to 9-26)	___	___	_____
12.	For major structure in high seismic risk area, has assessment been made of liquefaction potential of foundation soil during design earthquake (only loose saturated sands and silts are susceptible to liquefaction)? (See GEC No. 3, FHWA SA-97-076)	___	___	_____

*A response other than (yes) or (N/A) for any of these checklist questions is cause to contact the appropriate geotechnical engineer for a clarification and/or to discuss the project.

G. Structure Foundations – Driven Piles (Cont.)

Construction Considerations (Pgs. 9-4 to 9-35)	Yes	No	Unknown or N/A
13. Pile driving details such as: boulders or obstructions which may be encountered during driving; need for preaugering, jetting, spudding; need for pile tip reinforcement; driving shoes, etc.?	___	___	_____
14. Excavation requirements: safe slope for open excavations; need for sheeting or shoring; fluctuation of groundwater table?	___	___	_____
15. Have effects of pile driving operation on adjacent structures been evaluated such as protection against damage caused by footing excavation or pile driving vibrations?	___	___	_____
16. Is preconstruction condition survey to be made of adjacent structures to prevent unwarranted damage claims?	___	___	_____
17. On large pile driving projects, have other methods of pile driving control been considered such as dynamic testing or wave equation analysis?	___	___	_____

*A response other than (yes) or (N/A) for any of these checklist questions is cause to contact the appropriate geotechnical engineer for a clarification and/or to discuss the project.

GTR REVIEW CHECKLIST FOR DRILLED SHAFTS

H. Structure Foundations – Drilled Shafts (Pgs. 8-23 to 8-29)

In addition to the basic information listed in Section A, if drilled shaft support is recommended or given as an alternative, are conclusion/recommendations provided in the project foundation report for the following:

		Yes	No	Unknown or N/A
*1.	Are recommended shaft diameter(s) and length(s) for allowable design loads based on an analysis using soil parameters for side friction and end bearing?	___	___	_____
*2.	Settlement estimated for recommended design loads?	___	___	_____
*3.	Where lateral load capacity of shaft is an important design consideration, are p-y (load vs. deflection) curves or soils data provided in geotechnical report that will allow structural engineer to evaluate lateral load capacity of shaft?	___	___	_____
4.	Is static load test (to plunging failure) recommended?	___	___	_____
	Construction Considerations			
5.	Have construction methods been evaluated, i.e., can less expensive dry method or slurry method be used or will casing be required?	___	___	_____
6.	If casing will be required, can casing be pulled as shaft is concreted (this can result in significant cost savings on very large diameter shafts)?	___	___	_____
7.	If artesian water was encountered in explorations, have design provisions been included to handle it (such as by requiring casing and a tremie seal)?	___	___	_____
8.	Will boulders be encountered? (If boulders will be encountered, then the use of shafts should be seriously questioned due to construction installation difficulties and resultant higher cost to boulders can cause.)	___	___	_____

*A response other than (yes) or (N/A) for any of these checklist questions is cause to contact the appropriate geotechnical engineer for a clarification and/or to discuss the project.

GTR REVIEW FOR GROUND IMPROVEMENT TECHNIQUES

I. Ground Improvement Techniques

In addition to the basic information listed in Section A, if ground improvement techniques are recommended or given as an alternative, are conclusion/recommendations provided in the project foundation report for the following:

		Yes	No	Unknown or N/A
1.	For wick drains, do recommendations include the coefficient of consolidation for horizontal drainage, c_h, and the length and spacing of wick drains?	___	___	_____
2.	For lightweight fill, do recommendations include the material properties (ϕ, c, γ), permeability, compressibility, and drainage requirements?	___	___	_____
3.	For vibro-compaction, do the recommendations include required degree of densification (e.g., relative density, SPT blow count, etc.), settlement limitations, and quality control?	___	___	_____
4.	For dynamic compaction, do the recommendations include required degree of densification (e.g., relative density, SPT blow count, etc.), settlement limitations, and quality control?	___	___	_____
5.	For stone columns, do the recommendations include spacing and dimensions of columns, bearing capacity, settlement characteristics, and permeability (seismic applications)?	___	___	_____
6.	For grouting, do the recommendations include the grouting method (permeation, compaction, etc.), material improvement criteria, settlement limitations, and quality control?	___	___	_____

*A response other than (yes) or (N/A) for any of these checklist questions is cause to contact the appropriate geotechnical engineer for a clarification and/or to discuss the project.

GTR REVIEW CHECKLIST FOR MATERIAL SITES

J. Material Sites

In addition to the basic information listed in Section A, is the following information provided in the project Material Site Report.

		Yes	No	Unknown or N/A
1.	Material site location, including description of existing or proposed access routes and bridge load limits, if any?	___	___	_____
*2.	Have soil samples representative of all materials encountered during pit investigation been submitted and tested?	___	___	_____
*3.	Are laboratory quality test results included in the report?	___	___	_____
4.	For aggregate sources, do the laboratory quality test results (such as L.A. abrasion, sodium sulfate, degradation, absorption, reactive aggregate, etc.) indicate if specification materials can be obtained from the deposit using normal processing methods?	___	___	_____
5.	If the lab quality test results indicate that specification material cannot be obtained from the pit materials as they exist naturally, has the source been rejected or are detailed recommendations provided for processing or controlling production so as to ensure a satisfactory product?	___	___	_____
*6.	For soil borrow sources, have possible difficulties been noted, such as above optimum moisture content for clay-silt soils, waste due to high PI, boulders, etc.?	___	___	_____
*7.	Where high moisture content clay-silt soils must be used, are recommendations provided on the need for aeration to allow the materials to dry out sufficiently to meet compaction requirements?	___	___	_____
8.	Are estimated shrink-swell factors provided.	___	___	_____

*A response other than (yes) or (N/A) for any of these checklist questions is cause to contact the appropriate geotechnical engineer for a clarification and/or to discuss the project.

I. Material Sites (Cont.)	Yes	No	Unknown or N/A
*9. Do the proven material site quantities satisfy the estimated project quantity needs?	___	___	_____
10. Where materials will be executed from below the water table, have seasonal fluctuations of the water table been determined?	___	___	_____
11. Are special permit requirements been covered?	___	___	_____
12. Have pit reclaimation requirements been covered adequately?			
13. Has a material site sketch (plan and profile) been provided for inclusion in the plans, which contains:	___	___	_____
a. Material site number?	___	___	_____
b. North arrow and legal subdivision?	___	___	_____
c. Test hole or test pit logs, locations, numbers and date?	___	___	_____
d. Water table elevation and date?	___	___	_____
e. Depth of unsuitable overburden, which will have to be stripped?	___	___	_____
f. Suggested overburden disposal area?	___	___	_____
g. Proposed mining area and previously mined areas?	___	___	_____
h. Existing stockpile locations?	___	___	_____
i. Existing or suggested access road?	___	___	_____
j. Bridge load limits?	___	___	_____
k. Reclaimation details?	___	___	_____
14. Are recommended special provisions provided?	___	___	_____

*A response other than (yes) or (N/A) for any of these checklist questions is cause to contact the appropriate geotechnical engineer for a clarification and/or to discuss the project.

PS&E REVIEW CHECKLISTS

Plans and specifications (PS&E)** reviews of projects with major or unusual geotechnical features[1] should preferably be made by examining the plans, special provisions, and geotechnical report together.***

Subject — Page

SECTION A, General 31
SECTION B, Centerline Cuts and Embankments 32
SECTION C, Embankments Over Soft Ground 32
SECTION D, Landslide Corrections 33
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Certain checklist items are of vital importance to have been included in the PS&E. These checklist items have been marked with an asterisk (*). A negative response to any of these asterisked items is cause to contact the geotechnical engineer for clarification of this omission.

The information covered in Section A, General will apply to all geotechnical features. The rest of the sections cover additional important PS&E review items that pertain to specific geotechnical features.

** For purposes of this document, PS&E refers to a plan and specification review at any time during a project's development. Hence, the review may be at a preliminary or partial stage of plan development.

***When plan reviews are conducted at a partial stage the final geotechnical report may not be available.

[1]Major and unusual geotechnical features are defined in Table 1.

PS&E REVIEW CHECKLIST – GENERAL

A. General	Yes	No	Unknown or N/A
*1. Has the appropriate geotechnical engineer reviewed the PS&E to ensure that the design and construction recommendations have been incorporated as intended and that the subsurface information has bee presented correctly? <u>This is absolutely necessary</u>.	___	___	_____
2. Are the finished profile exploration logs and locations included in the plans?	___	___	_____
*3. Have geotechnical designs prepared by region or district offices or consultants been reviewed and approved by the State Headquarters' geotechnical engineer?	___	___	_____
4. Do the contract documents contain the special provisions as provided in the project geotechnical report?	___	___	_____
5. Have the following common pitfalls been avoided:			
a. Has an adequate site investigation been conducted (reasonably meeting or exceeding the minimum criteria given in Table 2)?	___	___	_____
b. Has the use of "subjective" subsurface terminology (such as relatively soft rock or gravel with occasional boulders) been avoided?	___	___	_____
c. If alignment has been shifted, have additional subsurface explorations been conducted along the new alignment?	___	___	_____
d. Has a note been included in the contract indicating all subsurface information is available to bidders?	___	___	_____
e. Do you think the wording of the geotechnical special provisions are clear, specific and unambiguous?	___	___	_____

*A response other than (yes) or (N/A) for any of these checklist questions is cause to contact the appropriate geotechnical engineer for a clarification and/or to discuss the project.

PS&E REVIEW CHECKLIST FOR SPECIFIC FEATURES

		Yes	No	Unknown or N/A
B.	Centerline Cuts and Embankments			
1.	Where excavation is required, are excavation limits and description of unsuitable organic soils shown on the plans?	___	___	_____
2.	Are plan details and special provisions provided for special drainage details, such as lined surface ditches, drainage blanket under sidehill fill, interceptor trench drains, etc.?	___	___	_____
3.	Are special provisions included for fill materials requiring special treatment, such as nondurable shales, lightweight fill, etc.?	___	___	_____
4.	Are special provisions provided for any special rock slope excavation and stabilization measures called for in plans, such as controlled blasting, wire mesh slope protection, rock bolts, shotcrete, etc.?	___	___	_____
C.	Embankments Over Soft Ground			
*1.	Where subexcavation is required, are excavation limits and description of unsuitable soils clearly shown on the plans?	___	___	_____
*2.	Where settlement waiting period will be required, has estimated settlement time been stated in the special provisions to allow bidders to fairly bid the project?	___	___	_____
*3.	If instrumentation will be used to control the rate of fill placement, do special provisions clearly spell out how this will be done and how the readings will be used to control the contractor's operation?	___	___	_____
4.	Do special provisions state that any instrumentation damage by contractor personnel will be repaired at the contractor's expense?	___	___	_____

*A response other than (yes) or (N/A) for any of these checklist questions is cause to contact the appropriate geotechnical engineer for a clarification and/or to discuss the project.

PS&E REVIEW CHECKLIST FOR SPECIFIC FEATURES

D.	Landslide Corrections	Yes	No	Unknown or N/A
1.	Are plan details and special provisions provided for special drainage details, such as lined surface ditches, drainage blankets, horizontal drains, etc.?	___	___	_____
*2.	Where excavation is to be made into the toe of an active slide, such as for a buttress or shear key, and stage construction is required, do the special provisions clearly spell out the stage construction sequence to be followed?	___	___	_____
*3.	Where a toe buttress is to be constructed, do the special provisions clearly state gradation and compaction requirements for the buttress material?	___	___	_____
*4.	If the geotechnical report recommends that slide repair work not be allowed during the wet time of the year, is the proposed construction schedule in accord with this?	___	___	_____

E.	Retaining Structures			
*1.	Are select materials specified for wall backfill with gradation and compaction requirements covered in the specification?	___	___	_____
2.	Are limits of required select backfill zones clearly detailed on the plans?	___	___	_____
3.	Are excavation requirements specified, e.g., safe slopes for excavations, need for sheeting, etc.?	___	___	_____
*4.	Where alternative wall types will be allowed, are fully detailed plans included for all alternatives?	___	___	_____
5.	Were designs prepared by the wall supplier?	___	___	_____
6.	Were wall supplier's design calculations and specifications reviewed and approved by the structural and geotechnical engineers?	___	___	_____

*A response other than (yes) or (N/A) for any of these checklist questions is cause to contact the appropriate geotechnical engineer for a clarification and/or to discuss the project.

		Yes	No	Unknown or N/A
E.	Retaining Structures (Cont.)			
*7.	Where proprietary retaining walls are bid as alternates, does bid schedule require bidders to designate which alternate their bid is for, to prevent bid shopping after contract award?	___	___	_____
8.	Have FHWA guidelines for experimental designations for certain proprietary wall types been followed?	___	___	_____
9.	Is ROW limit or easements shown on plans and mentioned in specifications where anchors are to be installed?	___	___	_____

Top-down Construction Type Walls (See "Manual for Design & Construction Monitoring of Soil Nail Walls", FHWA SA-96-069R and "Ground Anchors and Anchored Systems", FHWA IF-99-015)

		Yes	No	Unknown or N/A
*10.	For soil nail and anchor walls are the following included in the provisions:			
	a. Construction tolerances?	___	___	_____
	b. Minimum drill-hole size?	___	___	_____
	c. Material requirements?	___	___	_____
	d. Load testing procedures and acceptance criteria?	___	___	_____
	e. Construction monitoring requirements?	___	___	_____

*A response other than (yes) or (N/A) for any of these checklist questions is cause to contact the appropriate geotechnical engineer for a clarification and/or to discuss the project.

PS&E REVIEW CHECKLIST FOR SPECIFIC FEATURES

F.	Structure Foundations – Spread Footings	Yes	No	Unknown or N/A
*1.	Where spread footings are to be placed on natural soil, is the specific bearing strata in which the footing is to be founded clearly described, e.g., placed on Br. Sandy GRAVEL deposit, etc.?	___	___	_____
*2.	Where spread footings are to be placed in the bridge end fill, are gradation and compaction requirements, for the select fill and backfill drainage material, covered in the special provisions, standard specifications, or standard structure sheets?	___	___	_____

G.	Structure Foundations – Driven Piles			
1.	Do plan details adequately cover pile splices tip reinforcement, driving shoes, etc.?	___	___	_____
*2.	Where friction piles are to be driven in silty or clayey soils, significant setup or soil freeze affecting long-term capacity may occur. Do specifications require retapping the piles after 24 to 48 hour waiting period when required bearing is not obtained at estimated length at the end of initial driving?	___	___	_____
3.	Where friction piles are to be load tested, has a reaction load of four times design load been specified to allow load testing the pile to plunging failure so that the ultimate soil capacity can be determined?	___	___	_____
4.	Where end bearing steel piles are to be load tested, has load test been designed to determine if higher than 62 MPa (9 ksi) allowable steel stress can be used, e.g., 83 to 103 MPa (12 – 15 ksi)?	___	___	_____
*5.	Where cofferdam construction will be required, have soil gradation results been included in the plans or been made available to bidders to assist them in determining dewatering procedures?	___	___	_____

*A response other than (yes) or (N/A) for any of these checklist questions is cause to contact the appropriate geotechnical engineer for a clarification and/or to discuss the project.

G. Structure Foundations – Driven Piles (Cont.)	Yes	No	Unknown or N/A
*6. If a wave equation analysis will be used to approve the contractor's pile driving hammer, has a minimum hammer energy or estimated soil resistance in kN (tons) to be overcome to drive the piles to the estimated length, been given in the special provisions?	___	___	_____
*7. Has the appropriate safety factor, based on construction control method (static load test, dynamic load test, wave equation, etc.) been included? Have the specifications for the applicable construction control method been included?	___	___	_____

H. Structure Foundations – Drilled Shafts	Yes	No	Unknown or N/A
*1. Where drilled shafts are to be placed in soil, is the specified bearing stratum in which the drilled shaft is to be found clearly described, e.g., placed on Br. Sandy GRAVEL deposit, etc.?	___	___	_____
2. Where end bearing drilled shafts are to be founded on rock, has the rock elevation at the shaft pier locations been determined form borings at the pier locations?	___	___	_____
3. Where drilled shafts are to be socketed some depth into rock, have rock cores been extracted at depths to 3 m (10 ft) below proposed socket at location within 3 m (10 ft) of the shaft?	___	___	_____
*4. Are shafts equipped with PVC access tubes to accommodate non-destructive testing (gamma/gamma logging, cross-hole sonic logging) of the shaft? Are provisions for the appropriate non-destructive testing methods included?	___	___	_____

*A response other than (yes) or (N/A) for any of these checklist questions is cause to contact the appropriate geotechnical engineer for a clarification and/or to discuss the project.

PS&E REVIEW CHECKLIST FOR SPECIFIC FEATURES

I. Ground Improvement Techniques	Yes	No	Unknown or N/A
1. For wick drains, are contractor submittals required that include proposed equipment and materials, method(s) for addressing obstructions, and method(s) for splicing wick drains.	___	___	_____
2. For lightweight fill, are minimum/maximum densities, gradation, lift thickness, and method of compaction specified?	___	___	_____
3. For vibro-compaction, are contractor submittals required that include proposed equipment and materials? Are methods of measurement and acceptance criteria specified?	___	___	_____
4. For dynamic compaction:			
a. If method specification is used, are the following specified: tamper mass and size; drop height, grid spacing; applied energy; number of phases or passes; site preparation requirements; subsequent surface compaction procedures?	___	___	_____
b. If performance specification is used, are the following specified: minimum soil property value to be achieved and method of measurement; maximum permissible settlement?	___	___	_____
5. For stone columns, are the following specified: site preparation, backfill materials, minimum equipment requirements, acceptance criteria and quality assurance procedures?	___	___	_____
6. For grouting, are contractor submittals required that include proposed equipment and materials. Are methods of measurement and acceptance criteria specified?	___	___	_____

*A response other than (yes) or (N/A) for any of these checklist questions is cause to contact the appropriate geotechnical engineer for a clarification and/or to discuss the project.

PS&E REVIEW CHECKLIST FOR SPECIFIC FEATURES

J. Material Sites		Yes	No	Unknown or N/A
*1.	Is a material site sketch, containing the basic information listed on page 27, included in the plans?	___	___	_____
*2.	Has the material site investigation established a proven quantity of material sufficient to satisfy the project estimated quantity needs?	___	___	_____
3.	Where specification material cannot be obtained directly from the natural deposit, do the special provisions clearly spell out that processing will be required?	___	___	_____
4.	Are contractor special permit requirements covered in the special provisions?	___	___	_____
5.	Are pit reclaimation requirements clearly spelled out on the plans and in the special provisions?	___	___	_____

*A response other than (yes) or (N/A) for any of these checklist questions is cause to contact the appropriate geotechnical engineer for a clarification and/or to discuss the project.