Chapter Six
Substructure Design

6.1 INTRODUCTION

This chapter addresses substructure design for the construction of new structures.

6.1 SUBSURFACE INVESTIGATIONS

A proper design of a structure foundation requires thorough knowledge of the subsurface conditions at the structure site. The investigation should consist of subsurface investigation (borings, in situ testing, and sampling); laboratory testing; geotechnical analysis of all data; and design recommendations. The absence of a thorough geotechnical investigation or adequate data generally leads to:

- a foundation system with a large factor of safety, which is generally more expensive foundation or unsafe, and
- construction problems, disputes and claims.

6.1.1 INFORMATION NEEDED BY THE DESIGNER

The designer needs the following information from the subsurface investigation:

- The data to determine the bearing capacity of the soils and a prediction of the magnitude and rate of settlement. For granular soils, the bearing capacity can be obtained from the Standard Penetration Test (SPT) "N" values. Triaxial or unconfined compression testing and consolidation testing are required to determine the shear strengths and settlement parameters of cohesive soils. Undisturbed samples are needed for laboratory testing where compressible or organic soils are anticipated or found.
- Where rock is encountered, rock cores are retrieved to determine the quality of the rock. Using the Rock Quality Designation (RQD) and the rock type, the designer may estimate the ultimate compressive strength of the rock. (Refer to Section 10.6.2.3.2, Semiempirical Procedures for Bearing Pressure, *AASHTO Specifications*.) Rock cores may also be required to compute quantity estimates of rock excavation. (Rock is normally expected only in northern New Castle County.)
- Boring logs and soil classifications to provide a profile of the foundation soils.
- Water table elevations.

6.1.2 REQUESTS FOR BORINGS

The cost of a boring program is small compared to the overall structure cost. In the absence of adequate boring data, the design engineer must rely on extremely conservative designs with high safety factors.

The designer must request test borings from Materials and Research (M&R). A boring request form is shown as Figure 6-1. The request should be accompanied by:

- a location map showing the site with respect to the general area;
- a plan of the existing or proposed structure showing the approximate locations of the substructure units and the borings requested. The plan should show the existing and (if available) proposed right-of-way limits. When possible, location controls should be shown on the plan to assist the boring crew to accurately locate test holes by station and offset and to record ground surface elevations;
- depth of boring; and
- design schedule.

On complex projects, it may be more practical to hold a conference between the designer and M&R to discuss the scope and schedule of the proposed project. A two-stage boring schedule may be desirable for larger projects: an initial program followed later by an extensive program based on the results of the initial work. In preparing the request, the designer should consider the following requirements for borings:

- A minimum of one boring per substructure unit.
- A minimum of two borings per structure is required even where multi-plate and other large pipes are planned.
- Pier and abutment footings over 100 feet [30 m] in length require additional borings.
- The borings for adjacent footings should not be located in a straight line but should be staggered at the opposite ends of adjacent footings, unless multiple borings are taken at each footing.
- Where rock is encountered at shallow depths, additional borings or other investigation methods such as probes and test pits may be needed to establish the rock profile.
- Where muck is encountered at shallow depths, additional borings or other investigation methods may be needed to determine muck excavation quantities.
- The number of borings required and their spacing depend on the uniformity of soil strata and the type of structure. Erratic subsurface conditions require close coordination between M&R and the designer. Under non-uniform conditions, additional borings may be necessary.
- Where spread footings are being considered, the designer should request that the driller take continuous samples.

Figure 6-1
Boring Request Form

MEMORANDUM

To: Jim Pappas
Chief Materials and Research Engineer

From: Project Manager – Bridge Design

Date:

SUBJECT: Contract No. ______________________

Contract Title
P3e Project ID: __________________________

Attached for your use is a location map for the referenced project. Please provide this section with soil borings and an associated pavement design for this project.

Please contact __________________ at 760-______ if you have any questions. Thank you for your assistance.

Attachment
cc: (Assistant Director/Section Head)
(Squad Manager)
(Designer)
File

The designer should review the results of the test borings as soon as they are received to ensure that the borings are adequate and to give M&R as much time as possible to obtain any additional tests required. The designer must have soils information extending at least 10 feet [3 m] below the estimated pile tip elevation. To achieve this depth on the initial borings, borings should be at least 20 to 30 feet [6 to 9 m] below the top of the hard layer to ensure that the layer is of sufficient thickness. The hard layer is defined as having an N-value of 25 or more for 20 feet [6 m]. If the material is very soft above the hard layer, the boring should extend a minimum of 30 feet [9 m] below the top of the hard layer, unless a very hard layer is encountered. If the material gradually tightens up, 20 feet [6 m] may be sufficient. Where these precautions do not provide data to the appropriate depth, additional borings may be required.

6.1.3 MATERIALS AND RESEARCH SECTION TESTS

M&R routinely performs the following AASHTO tests for subsurface investigations at bridge sites:

- T88 Particle Size Analysis of Soils
- T89 Determining the Liquid Limit of Soils
- T90 Determining the Plastic Limit and Plasticity Index of Soils
- T100 Specific Gravity of Soils
- T206 Penetration Test and Split-Barrel Sampling of Soils
- T207 Thin-Walled Tube Sampling of Soils
- T208 Unconfined Compressive Strength of Cohesive Soils
- T216 One-Dimensional Consolidation Properties of Soils
- T225 Diamond Core Drilling for Site Investigation
- T265 Laboratory Determination of Moisture Content of Soils
- T267 Determination of Organic Content in Soils by Loss of Ignition
- T306 Progressing Auger Borings for Geotechnical Exploration
- ASTM D2850 Standard Test Method for Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils
- ASTM D4767 Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils

The Standard Penetration Test (SPT), T206, is the most commonly used test in subsurface investigations in Delaware. It is used to determine N-values. The N-values and other in-situ test results from the SPT provide an indication of soil density, consistency, friction angle ϕ, and shear strength. N values are commonly used for foundation design in granular soils. Laboratory testing of undisturbed samples (such as Tests T208, T216, D2850 and D4767) provides design parameters used for pile foundation design in cohesive soils and slope stability analyses. N-values are not recommended for pile design in cohesive soils. N-values must be corrected for effective overburden pressure in granular soils. See Figure 6-2 for the correction factors. (Whoever computes the bearing capacity should make the correction.) The relationship between the corrected N value and the angle of internal friction ϕ for granular soils is shown in Figure 6-3.

Figure 6-2a
N-Value Correction Factors for Granular Soils (U.S. Customary)

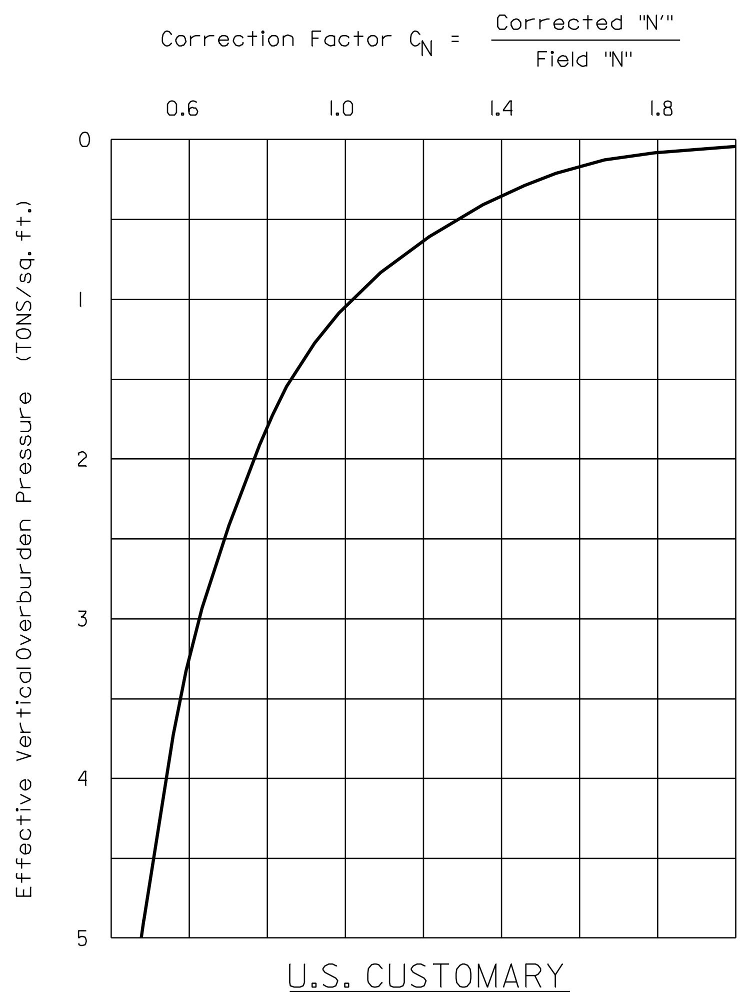


Figure 6-2b
N-Value Correction Factors for Granular Soils [Metric]

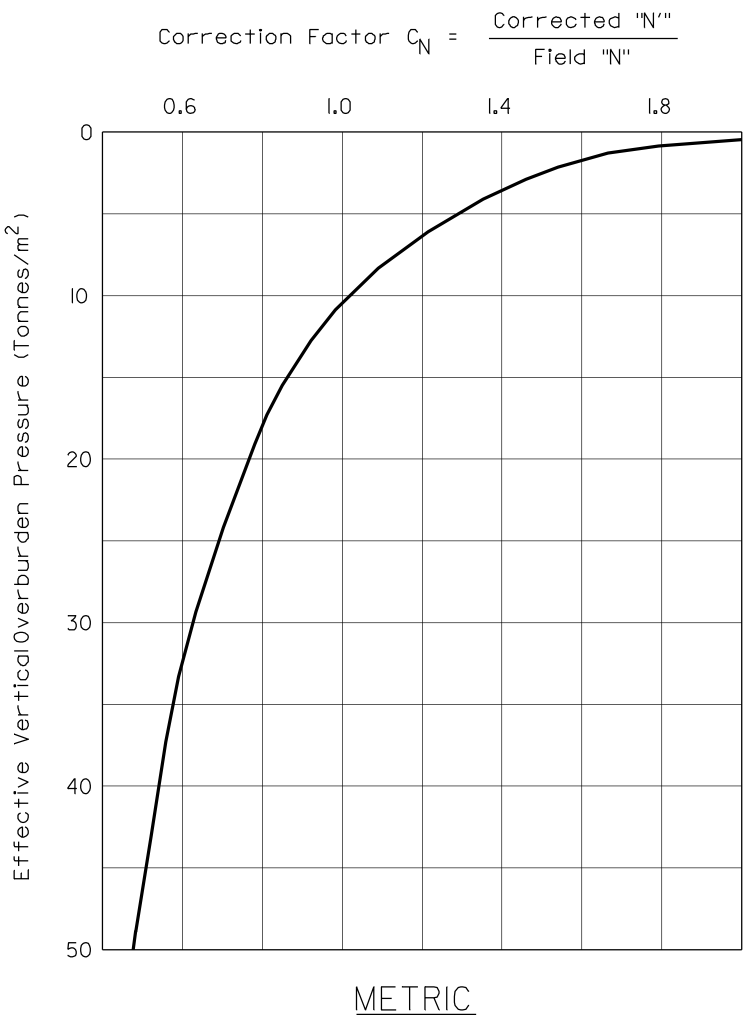
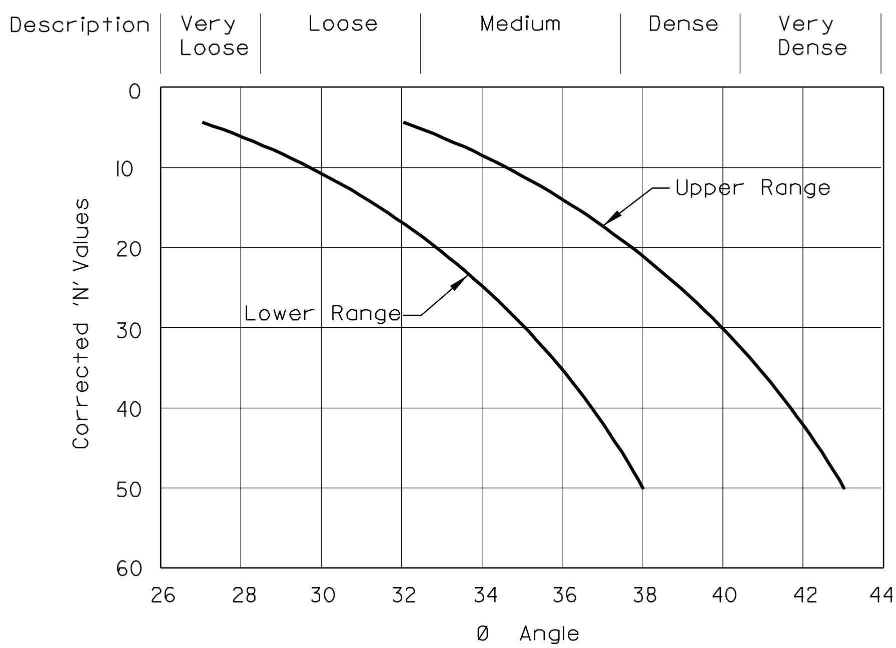


Figure 6-3
Relationship Between Corrected N-Values and ϕ



The equation for computing the correction factor is as follows:

$$C_N = 1 / (\text{Overburden Pressure})^{1/2}$$

where overburden pressure is in tonnes per square meter.

M&R can perform the following additional AASHTO tests if requested:

- T92 Determining the Shrinkage Factors in Soils, and
- T193 The California Bearing Ratio.

M&R is not equipped to run every test defined by AASHTO. However, private testing laboratories can be hired to perform other tests if warranted. Tests in this category include the following:

- T223 Field Vane Shear Test in Cohesive Soils - a test conducted in soft, saturated cohesive soils to determine the bearing resistance of the soil.
- T236 Direct Shear Test of Soils Under Consolidated Drained Conditions – a relatively simple test to measure the shear strength of fine granular soils. The test is not recommended for silts or clays because sample drainage cannot be controlled during the test.

The split spoon sampler used for the SPT provides disturbed samples. The SPT consists of driving a 2 inch [50.8 mm] O.D. split-spoon sampler into the soil with a 140 pound [63.5 kg] weight dropped 30 inches [762 mm]. The sampler is driven 18 or 24 inches [460 or 610 mm], and the blow count for each 6 inches [150 mm] is recorded. The number of blows for the second and third 6 inch [150 mm] increments of penetration is considered to be the penetration resistance, N. The SPT N-values can be used to estimate angle of internal friction and relative density of cohesionless soils. Static analytical design procedures for pile foundations use these design parameters.

Rock cores obtained from borings allow a qualitative evaluation of rock mass and distinguish between boulders and bedrock. Rock Quality Designation (RQD) values determined from cores are used in estimating the compressive strength and scourability of rock. Engineering classifications for in situ rock quality are shown in Figure 6-4. Rock Core Samplers (core barrels) are available in various diameters and length. DelDOT uses the NX core barrel, 2-1/8 inches [54 mm] inside diameter and 5 feet [1.5 m] in length.

Figure 6-4
Engineering Classification for In Situ Rock Quality

RQD%	Rock Mass Quality
90-100	Excellent
75-90	Good
50-75	Fair
25-50	Poor
0-25	Very Poor

To obtain the compressive strength of rock, refer to Section 10.6.2.3.2, Semiempirical Procedures for Bearing Pressure, of the *AASHTO Specifications*.

6.1.4 SOIL SAMPLES

Soil samples are divided into two categories: disturbed and undisturbed.

Disturbed samples have experienced large structural disturbances during sampling operations and may be used for identification and classification tests.

Undisturbed samples are those in which structural disturbance is kept to an absolute minimum. They are used for consolidation tests and strength tests, such as direct shear, triaxial shear, and unconfined compression. Strength tests provide shear strength design parameters which are used in static analysis for pile foundations and slope stability analyses. Consolidation tests provide coefficients of recompression (C_r), compression (C_c), and secondary compression (C_a) for estimating the magnitude of structural settlement, and also coefficients of consolidation (C_v), to determine the rate of settlement. Undisturbed samples are needed to evaluate cohesive soils parameters for estimating the magnitude and rate of settlement of spread footings or pile groups.

6.1.5 GROUND WATER MONITORING

Accurate ground water level information is needed for the estimation of soil densities, determination of effective soil pressures, and preparation of effective soil pressure diagrams. This information is vital for foundation design. Water levels will indicate possible construction difficulties that may be encountered during excavation and the level of dewatering effort required.

The ground water elevation will be recorded by the drillers when the borings are made. Since this elevation may vary throughout the year, the designer may request short- and long-term ground water elevation monitoring. Short-term monitoring is normally performed at 24-hour, 48-hour and 72-hour increments. Long-term monitoring will require installation of monitoring wells at the site.

6.1.6 BORING LOGS

A typical boring log is presented in Figure 6-5.

Information on the first sheet includes state and federal project numbers, the bridge number, the location of the boring, the surface elevation, the equipment used, the sampling method, the depth of investigation, and water level readings.

The other sheets of the log show the sample number, the sample depth, hammer blows per 6 inches [150 mm], descriptions of the material in the samples, the amount of material recovered in each sample, the laboratory soils classification, water table elevation and RQD results. The locations of undisturbed samples are designated with the sample numbers. Any other information is listed under “Remarks.”

A summary of soil analysis tests for the borings is shown in Figure 6-6.

The location (station and offset) and the approximate ground elevation at each boring should be recorded. The drill crew can determine the elevation by hand leveling from a centerline station or the deck grade (at an abutment) of an existing structure.

The boring data is entered into a graphics design file, using the Department's Boring Sheet program so designers can access it with CADD. The boring logs should be included in contract plans.

Figure 6-5a
Sample Boring Log

DELAWARE DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS
MATERIALS AND RESEARCH SECTION

PAGE 1 OF 4

F.A. Project:
Contract: 25-073-01 BRIDGE 3-362 ON S-465 @ CHIPMANS POND
Boring Loc.: STA. 4+55, 7' Lt. CENTERLINE

Boring No.: B # 1

Boring Surface Elev:			Reference:		
Wt. of Casing Hammer:		Lbs.	Average Fall:		IN.
Wt. of Sample Hammer:	140	Lbs.	Average Fall:	30	IN.
Type of: D-Sampler:	SPLIT-BARREL	O.D.	O.D. of SAMPLER:	2	IN.
S-Sampler:		O.D.	O.D.of SAMP.TUBE:		IN.
U-Sampler:		O.D.	O.D.of SAMP.TUBE:		IN.
Core Bit :		O.D.	O.D.of ROCK CORE:		IN.

Casing Size: 3 1/4" Inches; From Depth of: 0.0' To: 78.5'
HOLLOW STEM AUGER From Depth of: To:

Water Level Readings:

Date	Time	Depth of Hole	Depth of Casing	Depth of Water	Elev.of Water
/ /					
/ /					
/ /					
/ /					

Pay Quantities:
2 1/2 in. Dia.Dry Sample Boring: 80.0 Ft.; Dia. U-Sample Boring: Ft.
No. of 2 in. Dia. Shelby Tubes: ; No.of: U-Samples:
2 1/2 in. Dia.Contin.Sample Boring: Ft.; Core Drilling in Rock: Ft.

Boring Contractor: HILLIS-CARNES
Driller: K. HASTINGS
Helpers:

Remarks:

Reviewed By: RANDY FERGUSON Soils Supervisor: MAUREEN KELLEY

NOTES:
1. Make a separate log of each boring&each unsuccessful attempt. Keep a copy of all logs in the field.
2. In daily progress column indicate depth at beginning and end of work day, calendar date, time at beginning and end of work day and weather conditions.
3. All samples shall be numbered in consecutive order regardless of type; dry samples D, wash samples W, shelby tube samples S, undisturbed samples U. Do not assign numbers to lost samples but record blows and reasons for lack of recovery.
4. Mark each U-sample with boring number, sample number, depth, recovery and job number.
5. Record blows on sample per six inches of penetration. Note all blows and penetrations when taken at less than six inch intervals. Indicate method by which penetration of tube sampler was obtained.
6. Indicate changes of material in strata column and list generalized strata classifications.
7. List under remarks the manner by which changes in material were detected, all obstructions, any loss or gain of wash water including amount, the recovery of rock cores in feet and inches and percent of run, and any unusual occurences.

| BORING NUMBER: B # 1

Figure 6-5b
Sample Boring Log

STATE OF DELAWARE
DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS

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BORING NO. B # 1

CONTRACT: 25-073-01 BRIDGE 3-362 ON S-465 @ CHIPMANS POND
BORING LOCATION: STA. 4+55, 7' Lt. CENTERLINE

DAILY PROGRESS	NO.	SAMPLE DEPTH	BLOWS/6"	SAMPLE DESCRIPTION	CLASS/G.I.	REMARKS
8/18/04	1	0.0'	10	Moist dense brown silty fine to coarse sand	A-2-4 (0)	
			15	w/trace of gravel and clay.		
		1.5'	20			
				12" Recovery		
	2	2.5'	5	Wet stiff light brown fine sandy silt	A-4 (0)	
			4	w/trace of coarse sand and clay.		
		4.0'	7			
				16" Recovery		
	3	5.0'	4	Saturated medium dense light brown fine sand	A-3 (0)	
			7	w/some coarse sand, trace of silt and		
		6.5'	8	gravel.		
				18" Recovery		
	4	7.5'	4	Saturated medium dense light brown fine to	A-3 (0)	
			7	coarse sand w/trace of silt and gravel.		
		9.0'	10			
				18" Recovery		
	5	10.0'	2	Saturated very loose light gray fine sand	A-2-4 (0)	
			2	w/some silt, trace of coarse sand.		
		11.5'	1			
				18" Recovery		
	6	15.0'	2	Saturated loose light gray fine to coarse	A-2-4 (0)	
			3	sand w/some silt, trace of gravel.		
		16.5'	2			
				18" Recovery		
	7	20.0'	4	Saturated loose light gray fine to coarse	A-1-b (0)	
			4	sand w/some gravel, trace of silt.		
		21.5'	4			
				18" Recovery		
	8	25.0'	3	Saturated loose yellowish orange silty	A-2-4 (0)	
			4	fine sand w/trace of coarse sand.		
		26.5'	5			
				18" Recovery		

BORING NO. B # 1
SURFACE ELEV.

Figure 6-5c
Sample Boring Log

STATE OF DELAWARE
DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS

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BORING NO. B # 1

CONTRACT: 25-073-01 BRIDGE 3-362 ON S-465 @ CHIPMANS POND
BORING LOCATION: STA. 4+55, 7' Lt. CENTERLINE

DAILY PROGRESS	SAMPLE NO.	SAMPLE DEPTH	BLOWS/6"	SAMPLE DESCRIPTION	CLASS/G.I.	REMARKS
8/18/04	9	30.0'	2	Saturated medium dense brown fine sand	A-2-4 (0)	
			5	w/some coarse sand, gravel and silt.		
		31.5'	8			
				18" Recovery		
	10	35.0'	8	Saturated medium dense orange fine to	A-2-4 (0)	
			11	coarse sand w/some silt, trace of gravel.		
		36.5'	18			
				18" Recovery		
	11	40.0'	4	Saturated firm gray fine sandy silt w/trace	A-4 (0)	
			4	of clay, organic matter and gravel.		
		41.5'	4			
				18" Recovery		
	12	45.0'	3	Saturated very stiff gray fine sandy silt	A-4 (0)	
			4	w/some organic matter, trace of coarse sand.		
		46.5'	14			
				18" Recovery		
	13	50.0'	4	Saturated stiff gray clayey silt w/some	A-4 (7)	
			5	fine sand and clay, trace of organic matter.		
		51.5'	7			
				18" Recovery		
	14	55.0'	3	Saturated firm light gray silt w/some	A-4 (2)	
			3	clay, trace of fine sand.		
		56.5'	3			
				18" Recovery		
	15	60.0'	10	Saturated hard gray fine sandy clay	A-6 (6)	
			15	w/trace of coarse sand.		
		61.5'	25			
				18" Recovery		
	16	65.0'	7	Saturated very stiff light gray fine sandy	A-4 (0)	
			8	silt w/trace of coarse sand and clay.		
		66.5'	11			
				18" Recovery		

BORING NO. B # 1
SURFACE ELEV.

Figure 6-5d
Sample Boring Log

STATE OF DELAWARE
DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS

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BORING NO. B # 1

CONTRACT: 25-073-01 BRIDGE 3-362 ON S-465 @ CHIPMANS POND
BORING LOCATION: STA. 4+55, 7' Lt. CENTERLINE

DAILY PROGRESS	NO.	SAMPLE DEPTH	BLOWS/6"	SAMPLE DESCRIPTION	CLASS/G.I.	REMARKS
8/18/04	17	70.0'	14	Saturated dense gray silty fine sand.	A-2-4 (0)	
			21			
		71.5'	27			
				18" Recovery		
	18	75.0'	18	Saturated dense gray fine sand w/trace	A-3 (0)	
			24	of silt.		
		76.5'	20			
				18" Recovery		
	19	78.5'	24	Saturated dense gray fine sand w/trace	A-3 (0)	
			30	of silt.		
		80.0'	12			
				18" Recovery		
			(END)			

BORING NO. B # 1
SURFACE ELEV.

Figure 6-6a
Sample Summary of Boring Log Soil Analysis Tests

MATERIALS AND RESEARCH DIVISION
SUMMARY OF SOIL ANALYSIS TESTS
AASHTO TESTS:T-89,T-90, & T-265

PAGE 1

CONTRACT- 25-073-01
DATE----- SEPTEMBER 4, 2004

NAME--- BRIDGE 3-362 ON S-465
@ CHIPMANS POND

		*********** PERCENT PASSING ***********													
LOCATION	DEPTH	2.5	2	1	3/8	4	10	40	200	LL	PL	MO	OR	PI	CLASS	GI
B # 1 STA.	S#1 0.0- 1.5	100	100	100	100	98	91	65	27	18	15	8	--	3	A-2-4	0
4+55 7' Lt.	S#2 2.5- 4.0	100	100	100	100	100	100	97	44	15	14	13	--	1	A-4	0
CENTER LINE	S#3 5.0- 6.5	100	100	100	98	97	96	85	7	--	--	16	--	NP	A-3	0
	S#4 7.5- 9.0	100	100	100	98	97	95	54	9	--	--	18	--	NP	A-3	0
	S#5 10.0-11.5	100	100	100	100	100	100	97	20	--	--	30	--	NP	A-2-4	0
	S#6 15.0-16.5	100	100	100	100	100	99	69	15	--	--	30	--	NP	A-2-4	0
	S#7 20.0-21.5	100	100	100	93	91	89	50	10	--	--	23	--	NP	A-1-b	0
	S#8 25.0-26.5	100	100	100	100	100	100	99	24	--	--	34	--	NP	A-2-4	0
	S#9 30.0-31.5	100	100	100	94	88	81	62	15	--	--	25	--	NP	A-2-4	0
	S#10 35.0-36.5	100	100	100	95	94	91	52	12	--	--	21	--	NP	A-2-4	0
	S#11 40.0-41.5	100	100	100	100	100	100	99	39	20	17	23	3	3	A-4	0
	S#12 45.0-46.5	100	100	100	100	100	100	99	40	17	--	19	4	NP	A-4	0
	S#13 50.0-51.5	100	100	100	100	100	100	100	83	29	19	27	3	10	A-4	7
	S#14 55.0-56.5	100	100	100	100	100	100	100	93	24	20	27	--	4	A-4	2
	S#15 60.0-61.5	100	100	100	100	100	100	98	66	28	15	17	--	13	A-6	6
	S#16 65.0-66.5	100	100	100	100	100	100	99	50	23	22	20	--	1	A-4	0
	S#17 70.0-71.5	100	100	100	100	100	100	100	22	--	--	22	--	NP	A-2-4	0

Figure 6-6b
Sample Summary of Boring Log Soil Analysis Tests

MATERIALS AND RESEARCH DIVISION
SUMMARY OF SOIL ANALYSIS TESTS   PAGE   2
AASHTO TESTS:T-89,T-90, & T-265

CONTRACT-   25-073-01   NAME---   BRIDGE 3-362 ON S-465
DATE-----   SEPTEMBER 4, 2004   @ CHIPMANS POND

LOCATION	DEPTH	2.5	2	1	3/8	4	10	40	200	LL	PL	MO	OR	PI	CLASS	GI
		*********** PERCENT PASSING ***********														
	S#18															
	75.0-76.5	100	100	100	100	100	100	100	5	--	--	26	--	NP	A-3	0
	S#19															
	78.5-80.0	100	100	100	100	100	100	100	8	--	--	27	--	NP	A-3	0
	END															

6.1.7 SOILS DEFINITIONS

When selecting the type of foundation for bridges, the designer must consider the existing soils found at the site.

- Bedrock—The solid rock below the overburden soil, decomposed rock fragments or other loose superficial deposits. Bedrock exposed at the surface is known as rock outcrop.
- Boulders—Rounded fragments of rock that will be retained on a 3 inch [75 mm] sieve. Isolated boulders should not be confused with bedrock.
- Decomposed Rock—The upper portions of bedrock may be found in varying stages of decomposition. Decomposed rock represents the uppermost product of weathering and decomposition of bedrock in situ. In its most decomposed state, it can be compact soil that retains some of the appearance and texture of the original rock structure. Decomposed rock is not equivalent to bedrock for bearing capacity.
- Muck—Deposits of a saturated or unsaturated mixture of soils and organic matter not suitable for foundation material regardless of moisture content.
- Organic Matter—The more or less decomposed material of soil derived from organic sources, usually from plant remains. The term covers such material in all stages of decay.
- Organic Soils—Soils that contain significant amounts of muck.

While there are other soils classification systems, Delaware uses the AASHTO definitions to classify soils. The criteria for these classifications are in Figure 6-7.

- A-1 — Well-graded mixture of stone fragments or gravel, coarse sand, fine sand and a nonplastic or feebly plastic soil binder. This group also includes stone fragments, gravel, coarse sand, volcanic cinders, etc., without soil binder.
- A-1-a — Predominantly stone fragments or gravel, either with or without a well-graded binder of fine material.
- A-1-b — Predominantly coarse sand, either with or without a well-graded soil binder.
- A-2 — This group includes a wide variety of "granular" materials which are borderline between the materials falling in the A-1 and A-3 Groups and silt-clay materials of Groups A-4, A-5, A-6 and A-7. It includes all materials containing 35 percent or less passing the No. 200 [75 µm] sieve which cannot be classified as A-1 or A-3, due to fines content, plasticity or both, in excess of the limitations of those groups.
- A-2-4 and A-2-5 — Include various granular materials containing 35 percent or less passing the No. 200 [75 µm] sieve and with the portion passing the No. 40 [425 µm] sieve having the characteristics of Groups A-4 and A-5. These groups include such materials as gravel and coarse sand with silt contents or plasticity indexes in excess of the limitations of Group A-1, and fine sand with nonplastic silt content in excess of the limitations of Group A-3.
- A-2-6 and A-2-7 — Include materials similar to those in Subgroups A-2-4 and A-2-5 except that the fine portion contains plastic clay having the characteristics of Group A-6 or A-7.

Figure 6-7
AASHTO Soil Classification System

General Classification	Granular Materials (35% or less of total sample passing the 75 µm sieve)							Silty-clay Materials (More than 35% of total sample passing the 75 µm sieve)			
Group Classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5[1] A-7-6[2]
Sieve Analysis (% Passing)											
2.00 µm Sieve	50 max										
425 µm Sieve	30 max	50 max	51 max								
75 µm Sieve	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
For Fraction Passing No. 40 Sieve											
Liquid Limit (LL)				40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
Plasticity Index (PI)	6 max		Non-plastic	10 max	10 max	11 min	11 min	10 max	10 max	11 min	11 min
Usual Type of Material	Stone fragments, gravel and sand		Fine sand	Silty or clayey gravel and sand				Mostly silty soils		Mostly clayey soils	
Subgrade Rating	Excellent to good							Fair to poor			

[1] If PI $\leq$ LL – 30, it is A-7-5.

[2] If PI > LL – 30, it is A-7-6.

- A-3 — Typical material of this group is fine beach sand or fine desert blow sand without silty or clayey fines or with a very small amount of nonplastic silt. The group also includes stream-deposited mixtures of poorly graded fine sand and limited amounts of coarse sand and gravel. This material consists of nonplastic sands, such as beach sand, that is deficient in coarse material and soil binder.
- A-4 — Typically, this material is a nonplastic or moderately plastic silty soil having more than 35 percent passing the No. 200 [75 µm] sieve. The group also includes mixtures of fine silty soils with up to 36 percent silt and clay passing the No. 200 [75 µm] sieve.
- A-5 — This material is typically similar to Group A-4, except that it usually consists of fine sandy or silty soils containing mica or silica and may be highly elastic as indicated by the high liquid limit.
- A-6 — Typically, this material is a plastic clay soil having more than 35 percent passing the No. 200 [75 µm] sieve. The group also includes mixtures of fine clayey soils with up to 36 percent silt and clay passing the No. 200 [75 µm] sieve. Materials of this group usually have high volume changes between wet and dry states.
- A-7 — The typical material in this group is similar to Group A-6, except that it has the high liquid limit characteristics of Group A-5 and may be elastic as well as subject to high volume change.
- A-7-5 — Includes materials with moderate plasticity indexes in relation to liquid limit and which may be highly elastic as well as subject to considerable volume change.
- A-7-6 — Includes materials with high plasticity indexes in relation to liquid limit and which are subject to extremely high volume change.

The Group Index is an empirical constant based on the percent passing the No. 200 [75 µm] sieve, the liquid limit and the plasticity index of the material. The group index is shown in parentheses after the soils classification, e.g. A-7-6(12). In general, the higher the group index number, the poorer the soil bearing capacity.

6.1.8 GEOTECHNICAL REPORTS

Geotechnical reports are required for major structures or where foundation problems are anticipated. They shall include the following information:

- a summary of the findings;
- a plan view of the structure showing the location of the borings;
- the boring logs;
- an evaluation of the borings; and
- a foundation type recommendation.

The recommendation for all foundations should include the following:

- soil parameters, including depth, thickness and variability of soil strata, identification and classification of soils, shear strength, compressibility, stiffness, permeability, frost susceptibility, and expansion potential;
- rock parameters, including depth to rock, identification and classification of rock, rock quality (i.e., soundness, hardness, jointing, resistance to weathering, and solutioning), compressive strength, and expansion potential;
- presence of boulders, if encountered; and

- settlement considerations including required waiting period.

6.1.9 FOUNDATION REPORTS

Foundation reports are required for all structures. They shall include:

- the soil bearing capacity;
- the type of foundation;
- bottom footing elevation;
- settlement considerations including required waiting period;
- cofferdam requirements, if needed;
- any construction instrumentation and monitoring requirements;
- the anticipated scour depth; and
- slope stability.

If piles are recommended, the recommendations should also include:

- the type(s) of piles;
- the pile size;
- the design bearing capacity of the piles;
- the proposed pile lengths;
- the minimum pile tip elevation (even if it is higher than the final tip elevation); and
- the ultimate design pile capacity for drivability through the estimated scour layer. (See Section 6.2.2.4.3.)

Copies of the foundation report should be made available to Bridge Design, M&R, and FHWA (on non-exempt projects).

6.2 FOUNDATION DESIGN

A footing is the interfacing element between the superstructure and the underlying soil or rock. The loads transmitted from the superstructure to the underlying soil must not cause soil shear failure or damaging settlement. See Section 6.2.1 for limits of allowable total and differential settlement.

Structures may be widened on either the median side or outside on dual structures. Design and construction of the ultimate substructure will depend on available construction access and the estimated time when widening will be needed.

- If the structure will be widened on the median side (or on the outside where access will be limited), the ultimate substructure, including widened portions, should be built during the original construction.
- If the widening on the outside will be required in the immediate future (5 to 10 years), the substructure should be included in the original design, and may be built during the original construction.
- If widening is further in the future, the substructure should be designed to facilitate splicing the rebar and adding to the substructure.

It is essential to systematically consider various footing types and to select the optimum alternative based on the superstructure and subsurface conditions.

It is recommended to use the following approach to determine the optimum foundation alternative:

1. Select or assume bridge characteristics: pier type, super-structure and substructure types, span lengths, etc.
2. Determine the foundation loads to be supported and special requirements of the foundation. Special requirements may include limits on total and differential settlements, negative skin friction, lateral loads, scour, and time constraints on construction.

3. Evaluate the subsurface investigation and the laboratory testing data.
4. Prepare a final soil profile and critical cross sections on major projects. Determine soil layers suitable or unsuitable for spread footings or pile foundations.
5. Consider and prepare alternative designs, if feasible.
   - Shallow Foundations—spread footings
   - Deep Foundations—drilled shafts, pile foundations, or caissons

   Figure 6-8 shows foundation types and applicable soil conditions.
6. Prepare cost estimates for feasible alternative designs.
7. Select the optimum alternative. Generally the most economical alternative is selected and recommended. The ability of the local construction force, and availability of materials and equipment should also be considered.

Where the depth from the bottom of the footing to rock is minimal, since short piles are generally undesirable, the designer should specify excavation to rock rather than placing short driven piles. There are four options that preclude the use of short piles:

1. Specifying subfoundation backfill from the rock surface to the bottom of the footing.
2. Using subfoundation concrete instead of backfill where the depth to bedrock is shallow. Dimensions of the subfoundation concrete should be shown in the drawings.
3. Lowering the bottom of the footing (creating a thicker footing).
4. Constructing a taller pier or abutment.

Figure 6-8
Foundation Types and Applicable Soils Conditions

Foundation Type	Applicable Soils Conditions
Spread footing or wall footing	Any conditions where bearing capacity is adequate for applied load. May use on single stratum, firm layer over soft layer, or soft layer over firm layer. Check immediate, differential and consolidation settlements.
Pile foundation (friction, end bearing or combination)	Poor surface and near-surface soils. Soils of high bearing capacity 25 to 150 ft [7.5 to 45 m] below ground surface. Friction piles distribute load along pile shaft if the soil strength is adequate. End bearing piles transfer load by point bearing on dense soil or rock of high bearing capacity. Check settlement of pile groups in clay.
Caisson (drilled shaft) - generally end bearing or combination of end bearing and skin resistance	Poor surface and near-surface soils. Soil of high bearing capacity (point bearing) is 25 to 50 ft [7.5 to 15 m] below ground surface.

The designer must specify excavation into the rock to key the footing into the rock and to establish a suitable level bearing surface. The excavation in the rock can be the full width of the footing or can be benched.

Any footing that exceeds 3 feet [1 m] in depth must have vertical reinforcement to prevent cracking.

For major projects, if the estimated costs of feasible foundation alternatives (during the design stage) are within 15 percent of each other, then alternate foundation designs should be considered for inclusion in the contract documents.

6.2.1 SPREAD FOOTING FOUNDATIONS

It is necessary to consider the feasibility of spread footings in any foundation selection process. Spread footings are generally more economical than deep foundations (piles and caissons). Pile foundations should not be used indiscriminately for all subsurface conditions or for all structures. There are subsurface conditions where pile foundations are difficult to install and others where they may not be necessary. Depending on the subsurface investigation findings, spread footings may be used.

The term “bearing capacity” denotes the loading intensity that the bearing materials can sustain without such deformation as would result in settlement damaging the structure. Use the method defined in the *AASHTO Specifications* to compute soil bearing capacity.

In the design of continuous-span bridges, the designer must be aware of the possibility of settlement of the earth below footings. Check the soils report for types of soil that are prone to settle. If long-term differential settlement due to dead load is expected to exceed ½ inch [13 mm] or if total long-term settlement is expected to exceed 1 inch [25 mm], a pile foundation is required. Spread footings for continuous-span bridges may be used only with prior approval by the Bridge Design Engineer.

6.2.2 PILES

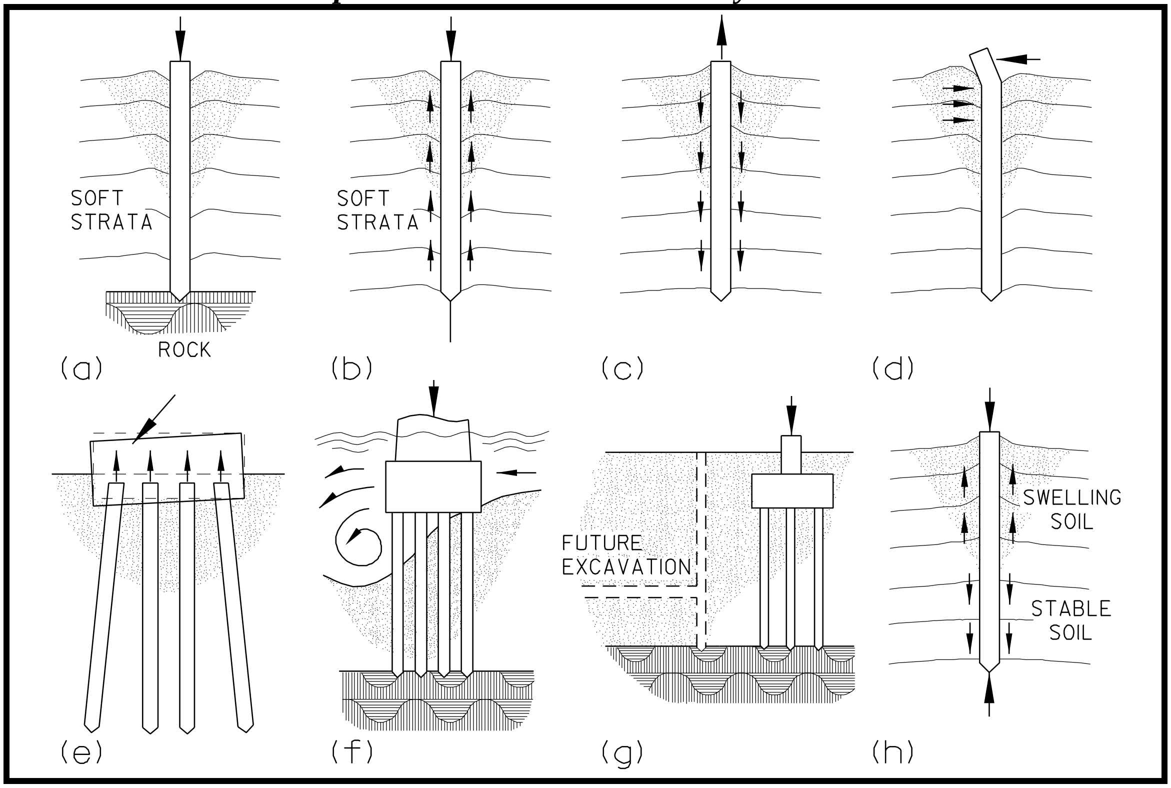
6.2.2.1 Determining the Need for a Pile Foundation

The first difficult problem facing the foundation designer is establishing whether the site conditions are such that piles must be used. Principal situations in which piles may be needed are shown in Figure 6-9.

Figure 6-9a shows the most common case in which the upper soil strata are too compressible or too weak to support heavy vertical loads. In this case, piles carry loads to a deep, rigid stratum and act as point bearing piles. In the absence of a rigid stratum within a reasonable depth, the loads must be gradually transferred, mainly by friction, along pile shafts-friction piles (Figure 6-9b). An important point to remember is that piles transfer load through unsuitable layers to suitable layers. The foundation engineer must define at what depth suitable soil layers begin in the soil profile.

Piles are frequently needed because of the relative inability of shallow footings to resist inclined, horizontal or uplift forces and overturning moments. Piles resist upward forces by downward skin friction around their shafts-uplift piles (Figure 6-9c). Horizontal forces are resisted either by vertical piles in bending (Figure 6-9d), or by groups of vertical and battered piles, which combine axial and lateral resistances of all piles in the group (Figure 6-9e).

Figure 6-9
Principal Situations Where Piles May Be Needed



Negative skin friction occurs where the soil surrounding the pile exhibits a downward movement with respect to the pile shaft. There are various causes of the downward movement of the soil. All the causes increase the load on the pile. This is also known as downdrag and is analyzed as additional axial load.

Pile foundations are often required where scour around footings could cause loss of bearing capacity at shallow depths (Figure 6-9f) or where future excavation is anticipated (Figure 6-9g). In either case the piles must be extended to develop the necessary capacity below the level of expected scour or excavation. This will prevent costly damage and eliminate the need for future underpinning.

Pile foundations may be needed in some areas to prevent undesirable seasonal movements of the foundations. Piles under such conditions are designed to transfer foundation loads, including uplift or downdrag, to a level unaffected by seasonal moisture movements (Figure 6-9h). See Section 6.2.2.4 for methods to reduce negative skin friction.

In many instances, either a shallow or pile foundation alternative is technically feasible. Under such circumstances, a reasonably detailed shallow foundation analysis including (1) dimensions and depth of shallow footings based on allowable bearing capacity, and (2) the magnitude and time-rate of settlement under anticipated loads, should be performed. A comparative analysis of the pile foundation alternatives should also be made. An approximate cost analysis of both alternatives should be performed and may include such factors as construction time and uncertainties. The emphasis in the selection of foundation type must be placed on the cost analysis of feasible alternatives.

Piles should not be used where the depth to bedrock is less than 10 feet [3 m]. In these cases, it is difficult to develop adequate lateral stability. See Section 6.2 for a discussion of alternatives to the use of short piles.

6.2.2.2 Types and Details

Load-bearing piles of various materials and design characteristics are commonly used. The types of load-bearing piles used in Delaware are:

- precast, prestressed concrete piles;
- precast-prestressed concrete cylinder piles;
- cast-in-place concrete piles;
- steel h-piles; and
- timber piles.

Load-bearing piles can also be classified by their method of load transfer from the pile to the soil mass, as shown in Figure 6-9. Load transfer can be by friction, point bearing or a combination.

6.2.2.2.1 Precast-Prestressed Concrete Piles

Precast-prestressed concrete piles are the preferred choice for use as pier bents over water. The minimum preferred size is 12 inches [300 mm] for abutments and 18 inches [450 mm] for bents.

Precast concrete piles are usually of constant cross section. Concrete piles are considered non-corrosive but can be damaged by direct chemical attack (e.g., from organic soil, industrial wastes or organic fills), electrolytic action (chemical or stray direct currents), or oxidation. Concrete can be protected from chemical attack by use of special cements or coatings.

Prestressed concrete piles are generally suitable for use as friction piles when driven in sand, gravel or clays. They are suitable for driving in soils containing boulders when designed for it. A rock shoe attached to the pile tip allows penetration through obstructions. Prestressed concrete piles are capable of high capacities when used as point bearing piles.

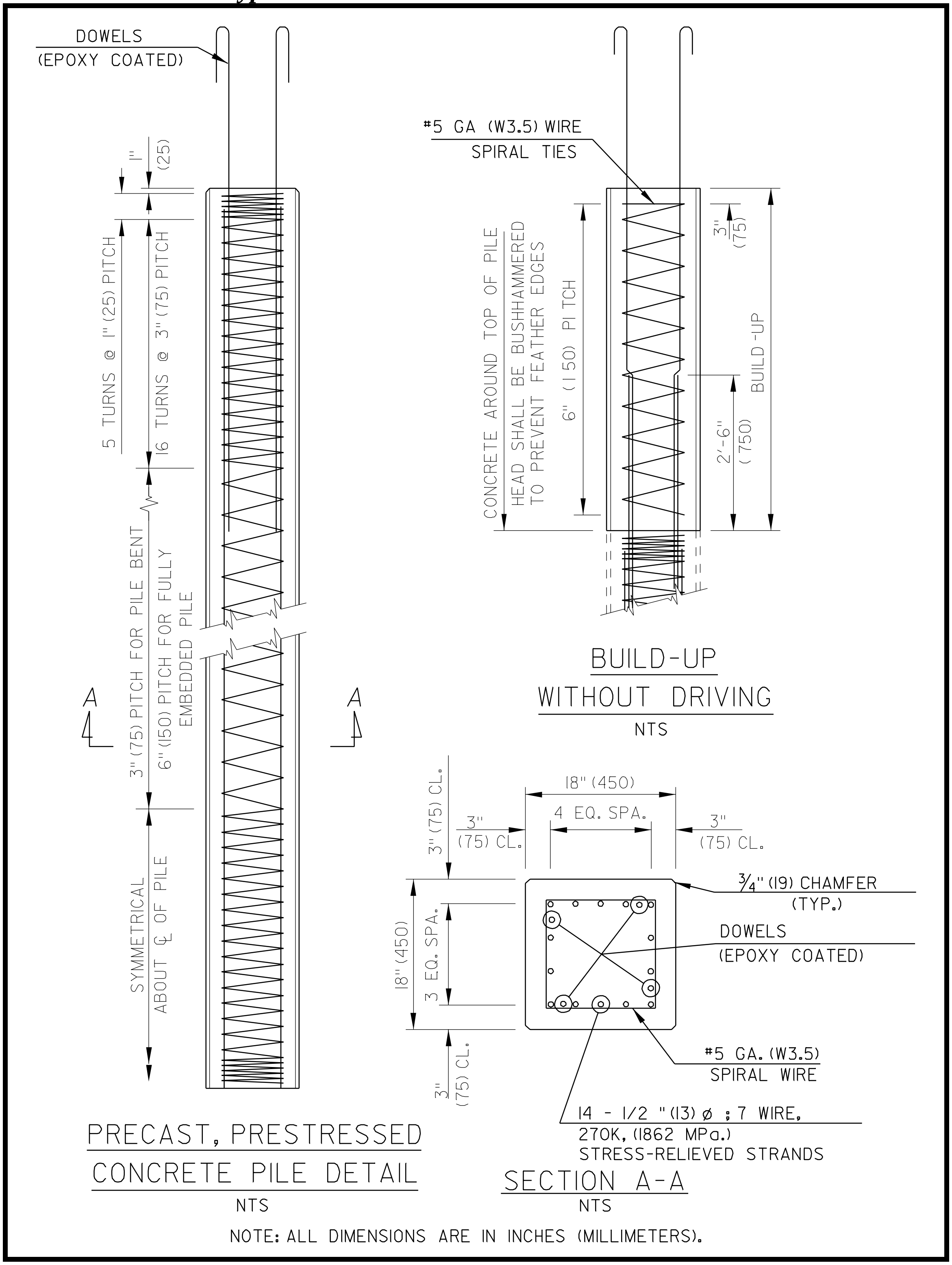
This pile consists of a configuration similar to a conventional reinforced concrete pile except that the longitudinal reinforcing steel is replaced by the prestressing steel. The prestressing steel is in the form of strands and is placed in tension. The prestressing steel is enclosed in a conventional steel spiral. In designing prestressed concrete piles for bents, the designer must specify special spiral reinforcement. Normal spiral reinforcement is used for piles fully embedded in soil.

Prestressed piles are pretensioned and are usually cast full length in permanent casting beds. Typical details for prestressed concrete piles with conventional spiral reinforcement are shown in Figure 6-10.

Dowel bars for pile cap development should only be used when necessary; when they are used:

- The holes for the dowels shall be field-drilled when possible.
- Preformed holes shall be long enough to allow for anticipated pile cut-offs.
- When preformed holes are used, the net cross-sectional area shall be considered in the wave equation analysis.
- The contractor shall submit a placement procedure for the grout which will have to be approved.

Figure 6-10
Typical Precast-Prestressed Concrete Pile Details



The primary advantage of prestressed concrete piles over conventional reinforced concrete piles is durability. Because the concrete is under continuous compression, hairline cracks are kept tightly closed and thus prestressed piles are usually more durable than conventionally reinforced piles. Another advantage of prestressing (compression) is that the tensile stresses which can develop in the concrete under certain driving conditions are less critical.

Splicing precast-prestressed concrete piles is not recommended. When piles must be driven to an elevation lower than the bottom of the cap to achieve bearing, cap heights may be increased to accomplish the design with approval of the Bridge Design Engineer.

6.2.2.2.2 Precast-Prestressed Concrete Cylinder Piles

Precast-prestressed concrete cylinder piles are post-tensioned piles cast in sections, bonded with a joint compound, and then tensioned in lengths containing several segments. Special concrete is cast by a process unique to cylinder piles which achieves high density and low porosity. The pile is virtually impervious to moisture. Cylindrical piles have good rigidity for long unsupported lengths. Results of chloride ion penetration and permeability tests on prestressed cylinder piles indicate that the spun cylinder piles have excellent resistance to chloride intrusion. Generally, cylinder piles are used for pile bents. The piles typically extend above ground and are designed to resist a combination of axial loads and bending moments. Diameters of 36 to 54 inches [900 to 1350 mm] may be used.

6.2.2.2.3 Cast-in-Place Concrete Piles

In general, cast-in-place concrete piles are installed by driving steel shells or pipes. Steel shell and pipe piles are easily spliced so predetermination of pile lengths is not as critical as for precast-prestressed concrete piles.

Reinforcing the pile length is required to provide adequate capacity. If the pile is fully embedded into soil, the minimum length of the pile reinforcement cage will be 6 feet [2 m]. In piles used for pile bents, the reinforcing cage must extend a minimum of 10 feet [3 m] below the point of fixity. See Section 6.5.3 for methods to determine the point of fixity. The designer must consider the relationship between pile reinforcement and the location of tapered pile sections. Normally, the reinforcement cage will not be tapered. The designer must properly select a tapered pile section when considering the termination point of the reinforcement cage.

Due to environmental and maintenance considerations, the designer should not specify cast-in-place piles for locations over water.

6.2.2.2.3.1 Steel Shell Piles

Cased fluted steel shell piles filled with concrete are the most widely used type of cast-in-place concrete pile.

After the shell has been driven and before concrete is placed, its full length is inspected internally. Reinforcing steel is required to provide a positive connection to the footing. Reinforcing steel may also be used to provide additional bending capacity. Shells are best suited for friction piles in granular material. Fluted steel shells are utilized in shell thicknesses of 3 gage to 7 gage [6.4 to 4.6 mm]. The fluted design has two primary functional advantages: 1) it adds the stiffness necessary for handling and driving lightweight piles; and 2) the additional surface area provides additional frictional resistance.

Splicing fluted steel shell sections is accomplished relatively easily by welding. A typical pile splice is shown on Figure 6-11.

Reinforcing steel is placed in the shells prior to placing the concrete. Figure 6-12 shows typical reinforcement details.

6.2.2.2.3.2 Steel Pipe Piles

Pipe piles usually consist of seamless, welded or spiral welded steel pipes. The pipe sizes used in Delaware are 12 to 18 inch [300 to 450 mm] diameters. The designer must specify the grade and thickness of steel for the pipe.

Pipe piles are driven with closed ends and are always filled with concrete. A closed-ended pile is generally formed by welding a flat plate of 0.5 to 0.75 inch [12.7 to 19.0 mm] or a conical point to the end of the pile. When pipe piles are driven to weathered rock or through boulders, a cruciform end plate or a conical point with rounded nose is often used to prevent distortion of the pile.

Pipe piles are spliced by using full penetration butt welds. The discussion presented under H-piles on corrosion is also applicable to pipe piles. A typical welded splice detail is shown in Figure 6-13.

Figure 6-11
Typical Splice Detail – Fluted Steel Shell

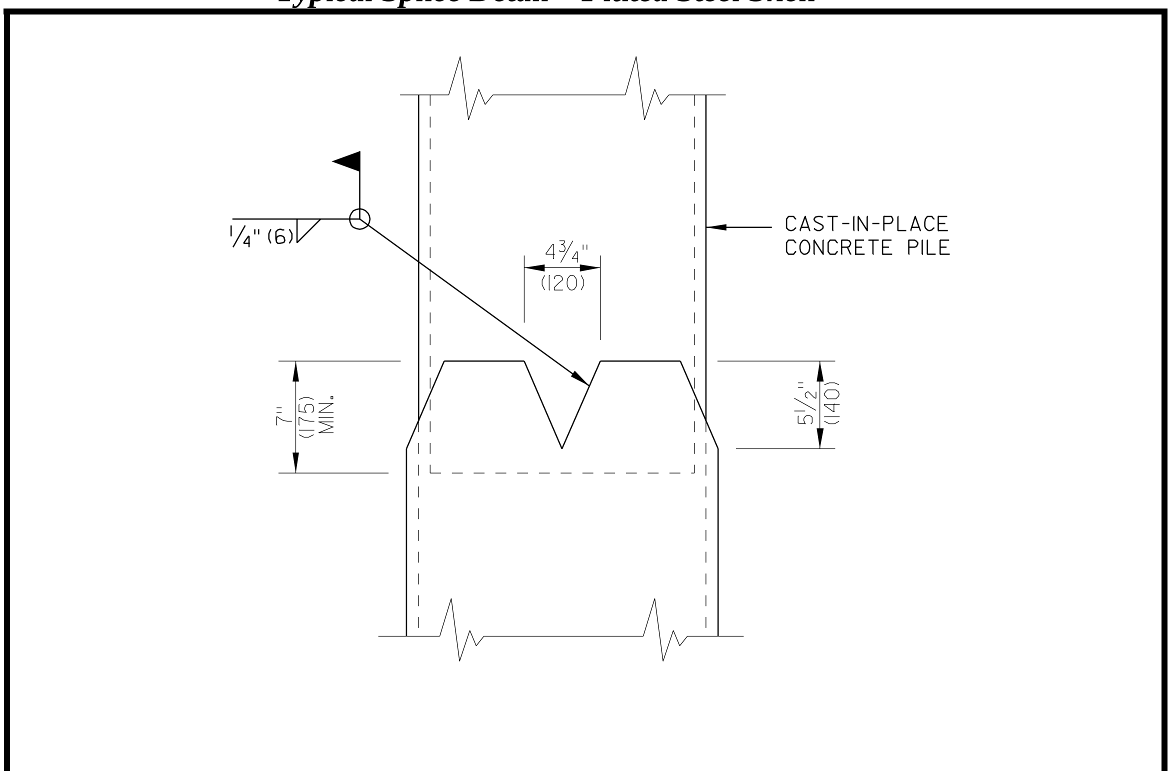
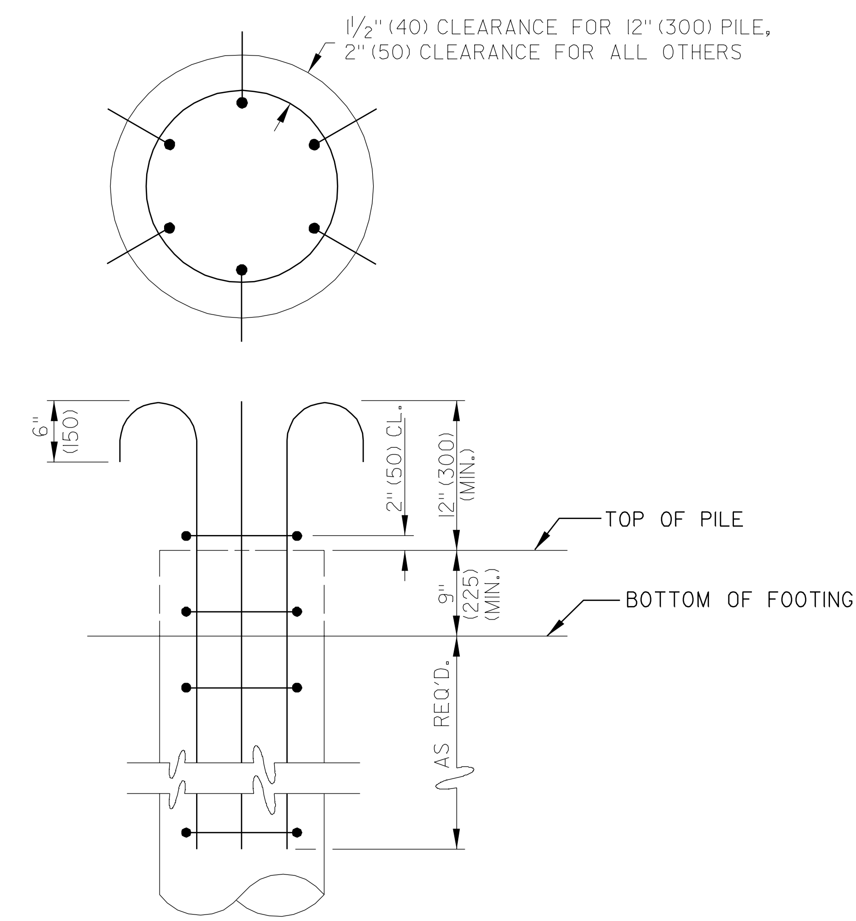
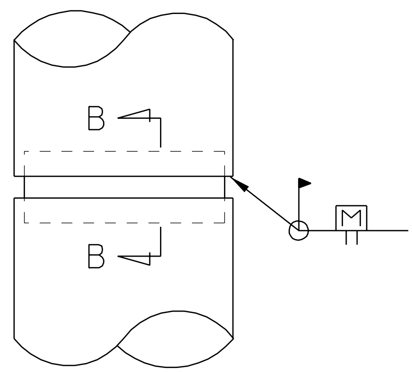


Figure 6-12
Typical Pile Reinforcement – Pile Shell



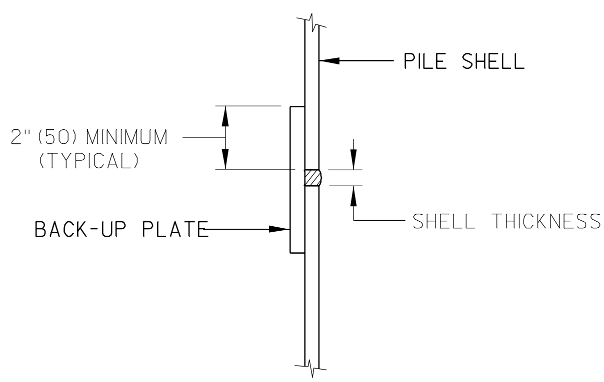
NOTE:
1. REBAR LAYOUT SHOWN FOR GENERAL LAYOUT.
2. THE ENGINEER SHALL BE RESPONSIBLE FOR THE PILE CAGE DESIGN.
3. NO PILE SPLICING TO BE ALLOWED ON ANY PORTION OF THE PILE THAT IS TO REMAIN EXPOSED IN THE COMPLETED STRUCTURE, EXCEPT IN SPECIAL SITUATIONS THAT MAY REQUIRE A SPLICE. ANY SPLICE SHALL BE APPROVED BY THE ENGINEER.
4. GAGE, ESTIMATED LENGTH, TAPER, SIZE AND REQUIRED DESIGN BEARING CAPACITY SHALL BE SPECIFIED BY THE ENGINEER.
5. NUMBER #4 (13) TIES MAY BE TACK WELDED ON EVERY OTHER BAR INSTEAD OF BEING TIED. SPIRAL TIES MAY BE SUBSTITUTED FOR TIES AT 1'-6" (450) SPACING.

Figure 6-13
Typical Splice Detail – Pipe Pile Shell



ELEVATION OF SPLICE
(USING ALL WELDED ALTERNATE)

SCALE : N.T.S.



NOTE:
BACK-UP PLATE TO BE CUT FROM SAME PILE SIZE AS IS BEING SPLICED. CUT AND BEND TO FIT INSIDE DIAMETER OF PILE.

SECTION B-B

NTS

NOTES:
1. NO PILE SPLICING TO BE ALLOWED ON ANY PORTION OF PILE THAT IS TO REMAIN EXPOSED IN COMPLETED STRUCTURE.
2. SPLICER SLEEVE MATERIAL SHALL BE STEEL CONFORMING TO ASTM DESIGNATION A709, GRADE 36 (250).

Steel pipe piles can be used as friction, point bearing or rock-socketed piles. They are commonly used where variable pile lengths are required because splicing is relatively easy. As noted in Section 6.2.2.1, piles should not be used where the depth to bedrock is less than 10 ft. [3 m].

6.2.2.2.4 Steel H-Piles

Steel H-piles consist of rolled wide flange steel H-sections. They are manufactured in standard sizes with nominal beam depths in the range of 8 to 14 inches [200 to 350 mm]. H-piles result in small relative volume displacement during driving, which may be advantageous when driving near other structures or buildings.

Steel H-piles commonly conform with AASHTO M183 [M183M] Specifications. They are not used where they will be exposed to the elements, so they are normally used only where fully embedded in soil to support footings, e.g. between footings and relatively shallow bedrock. As noted in Section 6.2.2.1, piles should not be used where the depth to bedrock is less than 10 ft. [3 m].

Splices are commonly made by full penetration butt welds. The splice should be as strong as the pile. Proprietary splices are also used for splicing H-piles.

A steel load transfer cap is not required if the top of the pile is adequately embedded in a concrete cap. The minimum embedment shall be 12 inches [300 mm]. Pile points are required for driving H-piles through dense soil or soil containing boulders. Pile points are also used for penetration into a sloping rock surface. Proprietary pile points welded to pile tips are commonly used. H-piles are suitable for use as end bearing piles, and occasionally as combination friction and end-bearing piles. Use of pile points must be approved by the Bridge Design Engineer. Because H-piles generally displace a minimum of material, they can be driven more easily through dense granular layers and very stiff clays. The problems associated with soil heave during foundation installation are often reduced by using H-piles. H-piles are commonly used for any depth because splicing is relatively easy.

6.2.2.2.5 Timber Piles

Timber piles are made from Southern yellow pine or Douglas fir trees. For hard driving, the tip should be provided with a metal shoe. See Section 618 of the *Standard Specifications* for minimum pile dimensions and straightness requirements.

Where a timber pile is subjected to alternate wetting and drying or is located in the dry above the water table, the service life may be relatively short due to decay and damage by insects. Even piles permanently submerged can suffer damage from fungus or parasites. Piling in a marine environment is also subject to damage from marine borers. Consequently, all timber piles specified for permanent structures will be treated. For the protection method, see Section 618 of the *Standard Specifications*. Other treatments specified by the American Wood Preserver's Association may be considered when approved by the Bridge Design Engineer. The designer should specify the desired treatment.

Driving timber piles often results in the crushing of fibers on the driving end (brooming). This can be controlled by using a driving cap with cushion material and metal strapping around the butt. Timber pile splices are not permitted.

Timber piles are best suited for use as friction piles in sands, silts and clays. They are not recommended to be driven through dense gravel, boulders, or till, or for end-bearing piles on rock since they are vulnerable to butt and tip damage in hard driving.

6.2.2.3 Selection of Pile Type

No exact criteria for selecting the various pile types can be given. Selection should be based on the factors in Figures 6-14 through 6-20. Figures 6-14 through 6-18 show design criteria for selected pile types. Figure 6-19 has pile type recommendations for various subsurface conditions. Figure 6-20 shows the placement effects of pile shape characteristics.

In addition to the considerations in the figures, the conditions posed by the specific project location and topography must be considered in the selection process. Two commonly encountered conditions are:

- Driven piles may cause vibration damage to adjacent structures and property.
- Waterborne operations may permit the use of longer pile sections because longer piles can be barged to the site.

Although one pile type may emerge as the only logical choice for a given set of conditions, more often several different types may meet all the requirements for a particular structure. In such cases, the final choice should be based on an analysis that assesses the costs of the alternatives, considering uncertainties in execution, local contractor experience, time delays, cost of load testing programs, as well as differences in the cost of pile caps and other elements of the structure. The cost analysis should be based on recent bid prices. For major projects, alternate foundation designs may be included in the contract documents if there is a potential for substantial savings.

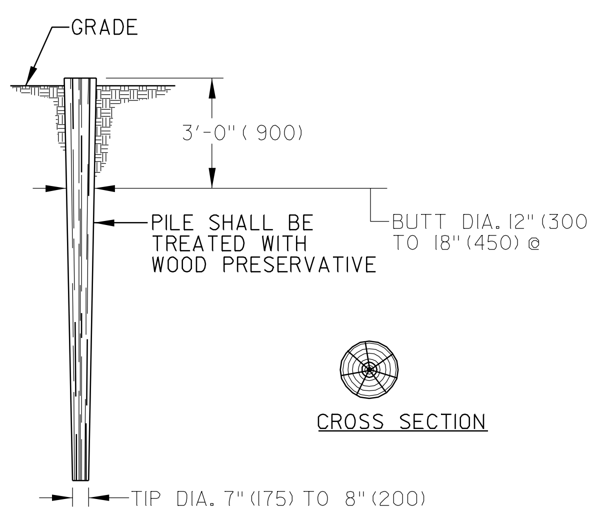
6.2.2.4 Pile Bearing Capacity

Generally, the pile length and bearing capacity are determined by the designer. See Section 10.7.3, Resistance at the Strength Limit State, in the *AASHTO Specifications* for the method of determining bearing capacity.

Where piles are driven through abutment fills, the resistance of the fill is not used in computing the pile bearing capacity. To eliminate driving resistance caused by fill material, auguring holes for piles through the fill depth should be required. The augured holes should be 2 inches [50 mm] less than the diameter of round piles or the diagonal dimension of square piles. Negative skin friction where the fill may settle can be prevented or accommodated by the following:

- The portion of the pile that will be in the fill area may be coated with bitumen to prevent bonding of the fill to the pile. This method cannot be used effectively if the pile has to penetrate a rigid stratum which could damage the coating during the driving.
- Pile size may be increased to accept the additional load.
- Pile may be driven in a preaugured hole of larger diameter than the pile shaft and the gap filled with bentonite slurry. Bentonite exhibits low friction characteristics and will limit the negative friction force.
- The number of piles may be increased to accommodate the increased load from negative skin friction.

Figure 6-14
Design Criteria for Piles – Timber

Considerations	**Criteria**
Length	30 – 60 ft. [9 – 18 m]
Material Specifications	ASTM D25
Maximum Stresses	
Design Loads	Up to 25 tons [225 kN] (compression only; do not include bending moment)
Disadvantages	• Difficult to splice • Vulnerable to damage from hard driving • Tip may have to be protected • Vulnerable to decay when piles are intermittently submerged
Advantages	• Comparatively low initial cost • Permanently submerged piles are resistant to decay • Easy to handle
Remarks	Best suited for friction pile in granular material
Typical Illustration	

Notes:

1. Lengths and loads indicated are for feasibility guidance only. They generally represent typical current practice.
2. Design load capacity should be determined by geotechnical engineering principles, limiting stress in piles, and type and function of structure.

Figure 6-15
Design Criteria for Piles – Steel H-Sections

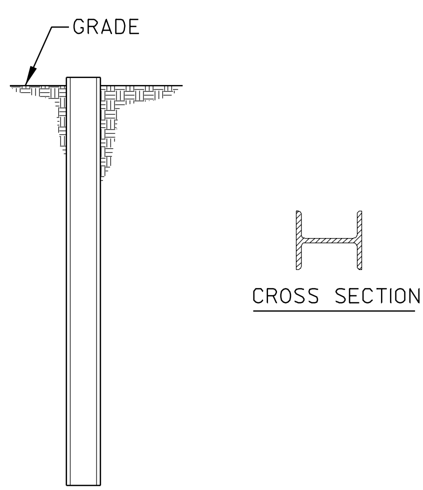
Considerations	Criteria
Length	40 – 100 ft. [12 - 30 m]
Material Specifications	ASTM A709 Grade 250
Maximum Stresses	
Design Loads	10 in [250 mm] = 55 tons [500 kN] 12 in [300 mm] = 65 tons [600 kN]
Disadvantages	• Vulnerable to corrosion where exposed • HP section may be damaged or deflected by major obstructions
Advantages	• Easy to splice • Available in various lengths and sizes • High capacity • Small displacement • Able to penetrate through light obstructions • Harder obstructions may be penetrated with appropriate point protection or where penetration of soft rock is required
Remarks	• Best suited for end bearing on rock • Reduce allowable capacity for corrosive locations
Typical Illustration	

Figure 6-16
Design Criteria for Piles – Precast-Prestressed Concrete

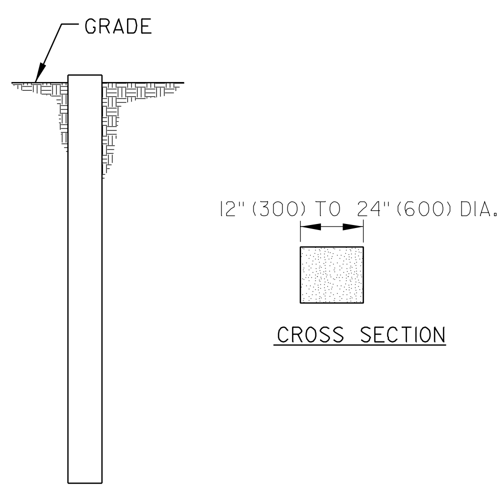
Considerations	Criteria
Length	60 – 100 ft. [18 - 30 m]
Material Specifications	ACI 318 for concrete AASHTO M31M for reinforcing steel
Maximum Stresses	
Design Loads	General loading range is 40 to 400 tons [355 to 3500 kN]
Disadvantages	• Relatively high breakage rate, especially when piles are to be spliced • Considerable displacement • Difficult to splice
Advantages	• High load capacities • Corrosion resistance can be attained • Hard driving possible
Remarks	Cylinder piles in particular are suited for bending resistance
Typical Illustration	

Figure 6-17
Design Criteria for Piles – Cast-in-Place Concrete

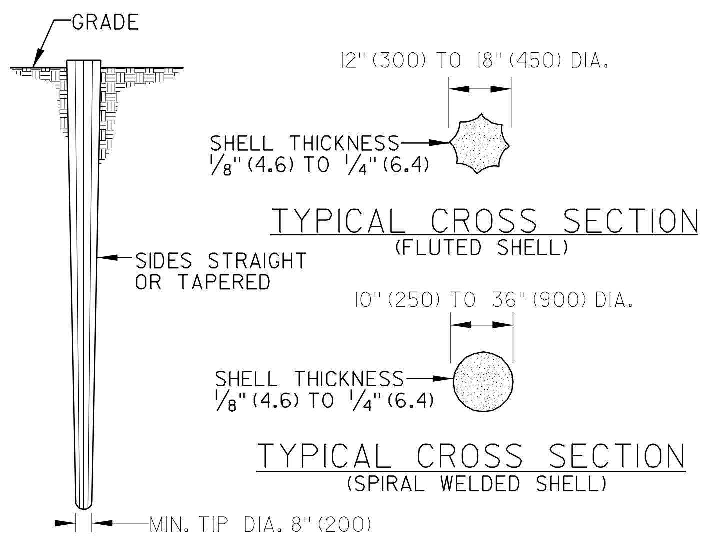
Considerations	**Criteria**
Length	30 – 80 ft [9 - 24 m]
Material Specifications	ACI 318 for concrete
Maximum Stresses	
Design Loads	12 in [300 mm] = 55 tons [500 kN] 14 in [350 mm] = 65 tons [600 kN] 15 in [400 mm] = 80 tons [700 kN]
Disadvantages	Considerable displacement
Advantages	• Can be re-driven • Shell not easily damaged
Remarks	Best suited for friction piles of medium length
Typical Illustration	

Figure 6-18
Design Criteria for Piles – Drilled Shafts

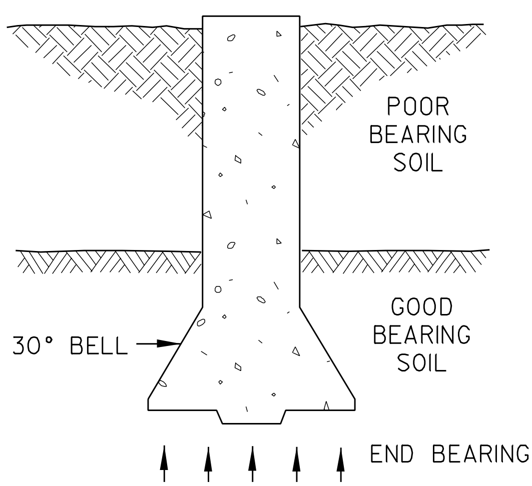
Considerations	**Criteria**
Length	Up to 100 ft [30 m]
Material Specifications	ACI 318 for concrete
Maximum Stresses	33% of 28-day strength of concrete
Design Loads	145 to 1000 tons [1300 to 9000 kN]
Disadvantages	• Construction procedures are critical to quality • Boulders can be a serious problem, especially in small diameter shafts
Advantages	• Economy • Complete nondisplacement • Minimal driving vibration to endanger adjacent structures • High skin friction • Good contact on rock for end bearing • Convenient for low-headroom underpinning work • Visual inspection of augered material • No splicing required • High bearing capacity • Availability of several construction methods • Can be continued above ground as a column
Remarks	• Best suited as a high capacity end bearing pile • Suited for installation in stiff clays
Typical Illustration	

Figure 6-19
Guide to Pile Type Selection for Subsurface Conditions

Typical Problem	Recommendations
Boulders overlying bearing stratum	Use heavy nondisplacement pile with a point and include contingent pre-drilling bid item in contract.
Loose cohesionless soil	Use tapered pile to develop maximum skin friction.
Negative skin friction	Use smooth steel pile to minimize drag adhesion. Avoid battered piles. Use bitumen coating for piles.
Deep soft clay	Use rough concrete piles to increase adhesion and rate of pore water dissipation.
Artesian pressure	Caution required when driving thin-wall pile shells due to potential collapse of shell from hydrostatic pressure. Pile heave common to closed-end piles.
Scour	Do not use tapered piles unless large part of taper extends well below scour depth. Design permanent pile capacity to mobilize soil resistance below scour depth.
Coarse gravel deposits	Use prestressed concrete piles where hard driving is expected in coarse soils. Use of H-piles in these deposits often results in excessive pile lengths.

Figure 6-20
Guide to Pile Shape Effects

Shape Characteristics	Pile Types	Placement Effect
Displacement	Closed-end steel pipe pile and precast-prestressed concrete	Density cohesionless soils, remold and temporarily weaken cohesive soils. Setup time or freeze for large pile groups in sensitive clays may be up to six months.
Nondisplacement	Steel H-pile	Minimal disturbance to soil.
Tapered	Timber and fluted steel shells	Increased densification of soil, high capacity for short lengths in granular soils.

6.2.2.4.1 Resistance Factors

Resistance factors are selected based on the method of design, with modifications based on the method of controlling installation. See Section 10.5.5, Resistance Factors in the *AASHTO Specifications*.

6.2.2.4.2 Lateral Loads on Piles

In addition to axial loads, piles are expected to transmit lateral loads into the soil. This causes both shearing forces and bending moments in the pile. The designer must evaluate pile capacity considering the axial load, the lateral loads and the combination of these loads. Lateral deflections and displacement of the pile axis must be evaluated simultaneously. These lateral loads must be resisted by the stiffness of the pile or pile group through mobilization of resistance in the surrounding soil as the pile deflects. Battered piles should be evaluated for resistance of lateral loads. Any excess lateral loads then may be resisted by the vertical piles as specified below.

The lateral load should be analyzed in accordance with Section 10.7.3.8, Lateral Load, in the *AASHTO Specifications*.

6.2.2.4.3 Static Analysis

The static load capacity of a pile can be defined as the capacity of the ground to support the loads on the pile imposed by the structure. A static analysis is performed to determine the static load capacity (ultimate and design capacities) of individual piles and of pile groups. Static pile capacity computations are necessary to estimate the number of piles and the required pile lengths both for the design of substructure elements and for ordering piles of the correct length from the supplier.

The ultimate static load capacity of an individual pile and of a pile group is the smaller of:

- the structural capacity of the pile(s) or
- the geotechnical capacity of the pile(s) (i.e., the ability of the pile or pile group to transfer load to the soil medium by friction, end bearing or a combination of both).

The capacity of the soil medium surrounding the pile can be estimated by geotechnical engineering analysis using:

- the dynamic penetration test data (i.e., Standard Penetration Test) which is used most frequently or
- shear strength parameters of the soil surrounding the pile for cohesive soils.

The designer selects the type of pile needed based on the foundation report. A static analysis is performed to determine the length of piles required. The unsupported length of pile (assuming scour due to the Q_{100} storm occurred) is checked for bending. From the analysis, the ultimate capacity of the piles is checked.

Generally, two static analyses are required for a design: one to determine the number and depth of piles; and one to evaluate pile-driving resistance. The first analysis does not consider the soil located above the calculated depth of scour in determining pile capacities. The second analysis is used to evaluate the pile-driving resistance in order to establish the necessary capability of the driving equipment. In this analysis, the driving resistance from both the unsuitable soil and soil in the scour prism is considered. The first static analysis must include the appropriate resistance factor for the Q_{100} condition. The ultimate capacity for the Q_{500} condition shall be checked for the extreme event limit state.

As geotechnical analysis results are received, the pile design is analyzed by one of the following methods:

- The Nordlund Method is used for non-cohesive soils-sands and gravels. Refer to the *FHWA Manual on Design and Construction of Driven Pile Foundations*, page 141 for the method and page 177 for example calculations.

  The ultimate bearing capacity (Q_u) of a pile in homogeneous soil may be expressed by the sum of point resistance Q_p and skin resistance Q_s, or

  $$Q_u = Q_p + Q_s = q_p A_p + f_s A_s$$

  Where:
    - q_p equals the unit bearing capacity of the pile point of area A_p, and
    - f_s equals the average unit skin friction on the pile shaft of area A_s.
- For cohesive soils use the Tomlinson Method or Beta Method. Refer to pages 189 and 192, respectively, of the *FHWA Manual on Design and Construction of Driven Pile Foundations*. These methods provide satisfactory results for soft to medium clays but are not reliable for stiff clays. The use of load tests should be considered for piles in stiff clays, even if the number of piles may be less than the minimum number of piles where load tests are normally justified.

The computer program DRIVEN is available for these methods. See Chapter 12 for details.

6.2.2.4.4 Dynamic Formula

The simplest forms of dynamic formulas in use today are based on equating the energy used to the work done (i.e., a pile is moved a certain distance against the soil resistance).

Statistical comparisons have shown poor correlations and wide scatter between formula predictions and actual load tests. As a result, it is recommended that use of dynamic formulas be limited to cases where well-supported empirical correlations under a given set of physical and geological conditions are available.

The one-dimensional wave equation analysis has eliminated many shortcomings associated with dynamic formulas by providing a more realistic analysis of the pile-driving process. Wave equation analysis uses wave propagation theory to monitor the longitudinal wave transmitted along the pile axis when it is struck by a hammer's impact. As the ram impact occurs, a force pulse is developed in the pile that travels downward toward the pile tip at a constant velocity, which depends on the pile material properties. When the force pulse reaches the embedded portion of the pile, it is attenuated by soil frictional resistance along the pile. If the attenuation is incomplete, the force pulse will penetrate into the soil when the peak force generated reaches the pile tip and a reflected force pulse governed by the soil tip resistance is generated. The pile will be driven into the soil when the ram impact exceeds the ultimate soil resistance at the pile tip.

The wave equation analysis provides the following information for a given pile length penetration:

- a relationship between ultimate pile capacity and blow count per foot [meter];
- actual driving stresses in the pile and penetration;
- information to evaluate the hammer performance; and

- a determination of energy output, the stroke, and the speed of hammer in blows per minute.

6.2.2.4.4.1 GRLWEAP

DelDOT utilizes GRL's Wave Equation Analysis of Pile Driving (WEAP) for pile foundation analysis and design. This program simulates a foundation pile under the action of an impact pile hammer. The program computes the following:

- The blow count (number of hammer blows per unit length of permanent set) of a pile under one or more assumed ultimate resistance values and other dynamic soil resistance parameters, using an assumed hammer and driving system (helmet, hammer cushion, pile cushion). It is important to use appropriate, available hammers and driving system elements commonly used by DelDOT contractors.
- The axial stresses in a pile corresponding to the computed blow count.
- The energy transferred to a pile.

Based on these results, the following can be indirectly derived:

- the pile's bearing capacity at the time of driving or restriking, given its penetration resistance (blow count),
- the pile stresses during driving, and
- the expected blow count based on the computed static bearing capacity of the pile.

By considering various hammer types, driving system parameters and pile properties during a number of simulations, an optimal system can be selected.

6.2.2.4.4.2 Input Information

GRLWEAP requires input information about the pile hammer, the pile-driving system, the pile and the soil. Hammer data for most commonly encountered hammers are in the program data files. The pile driving system consists of the hammer driving cushion, helmet (including striker plate, inserts and adapters), and pile cushion (for concrete piles). Pile data required consists of total length, cross sectional area, elastic modulus, and specific weight as a function of depth. In most cases, these values are constant. Soil data should include SPT values and the assignment of damping and quake values.

6.2.2.4.4.3 Damping and Quake

Quake is the elastic compression of soil during pile driving. Damping is the loss of driving energy due to inelastic soil response. Both quake and damping occur on the surface (skin) and at the toe of the pile. Quake and damping factors suitable for preliminary analyses are included in the GRLWEAP Manual. Use Smith damping factors in most cases.

6.2.2.4.4.4 Output

Three types of output are available from the WEAP program:

- a printout that allows checking the input data;
- the actual results of the analysis; and
- program messages, including warnings, interrupt information and general information.

As with any design, the designer should be thoroughly familiar with the structural theory and the software being used. The current GRLWEAP program reduces the amount of guessing required while entering the data while increasing the options available for selection. This provides the engineer with significant freedom in modeling during design with the intent of simulating actual field conditions. The following should be remembered in using the program:

- Bearing capacity predictions require the observation of blow counts. Wave equation results plus observed blow counts yield a bearing capacity that reflects the soil strength “at the time of testing.”
- For fluted steel pile shells, the residual stress analysis (RSA) option is recommended. This will tend to make the predicted stresses and capacities higher. Underpredictions are, however, still anticipated, particularly when end-of-drive field blow counts are used instead of beginning-of-restrike blow counts.
- Good capacity correlations require restrike information.
- Average hammer performance parameters included in the hammer data file may overestimate or underestimate the actual hammer output, yielding either low or high predictions of stresses and bearing capacities.
- Properly applied, the predicted results should have an error less than 20 percent unless the pile soil system fails with different mechanisms under the static and dynamic loads.

For a complete explanation of Wave Equation theory, including computer program use, documentation and sample computations, refer to the FHWA *Manual on Design and Construction of Driven Pile Foundations* as well as GRLWEAP: Wave Equation Analysis of Pile Driving.

6.2.2.4.5 Pile Driving Analyzer

The pile driving analyzer (PDA) is an instrument which monitors the force and acceleration at the pile head during driving through the use of strain gages and accelerometers attached to the pile. Force and acceleration measurements taken near the top of a pile during impact provide the necessary information for determining:

- Bearing capacity
- Hammer performance
- Maximum driving stresses
- Pile or shaft integrity

The Department utilizes the PDA for test piles during initial drive and restrike. The PDA provides the data listed above in the field while the pile is being driven. The data from the PDA is also utilized to run the CAPWAP computer program. This program obtains a “best possible match” between measured and computed pile driving variables. The result is a more accurate prediction of bearing capacity, driving stresses and distribution of resistance.

The PDA and CAPWAP information collected during construction shall be used by the designer for production pile driving criteria.

6.2.2.4.6 Pile-Soil Interaction

Load transfer is achieved in pile foundations through pile end bearing, soil/pile friction transfer, or a combination of these two. Prior to considering the static design methods in current use for estimating pile capacity it is desirable to review: (1) events that occur in the pile-soil system during and after driving; and (2) load-transfer mechanisms.

The designer will also utilize the wave equation to evaluate pile stresses during driving, pile capacity, pile hammer capacity, driving resistances, pile tip elevations, and safe bearing values. Each use of the wave equation is good for a specific embedment length. The analysis should consider several conditions which will occur during driving:

- 5% initial driving of precast-prestressed concrete piles;
- 70% penetration;
- 90% penetration;
- 100% penetration (estimated tip elevation); and
- 110% penetration.

The analysis for the initial driving phase must include a check for tensile stresses on the pile. The occurrence of this condition will depend on the foundation soils. The Geotechnical and Design report should address this disturbance if it is pertinent.

In the case of a pile foundation, the soil below the pile cap is almost always disturbed. The degree of disturbance depends on soil type and the methods of pile placement.

Sometimes bearing is not attained at the planned pile depth. When this occurs with cast-in-place or H-piles, the piles are spliced and driving continues until the specified bearing is reached.

It is difficult to splice precast concrete or timber piles. Where bearing is not attained at the planned depth for these types of piles, driving should stop with sufficient pile length above the ground line to permit development of additional skin friction through soil "freeze" to gain pile bearing. The contractor should be required to wait at least 48 hours, or as specified by the Bridge Design Engineer, for the freezing to occur before restriking. Where a small number of piles is involved, it is better to splice and continue driving rather than waiting for freeze to occur. Restriking is permitted in soils with cohesive properties. The maximum total number of restrike hammer blows will be 20 or a maximum total penetration of 6 inches [150 mm], whichever occurs first. The hammer should be warmed up on another pile before restriking.

Restriking is the act of driving a previously driven pile at some time after initial installation was completed, usually to evaluate the occurrence of freeze or relaxation.

Freeze is the restoration of the shear strength exhibited by soils after being remolded and disturbed by pile driving which results in an increase in load-carrying capacity after driving or during interruptions in driving. This is due to changed conditions such as soil pore-water pressure changes, soil remolding, and stress redistributions in the soil.

Relaxation is the characteristic of a pile to show a decreased static capacity after driving due to changed conditions such as soil pore-water pressure changes, soil remolding, and stress redistributions in the soil.

6.2.2.4.7 Pile Load Tests

Load testing is the most accurate method of verifying pile capacity. The designer must specify the type of load test to be used. Conventional load test types are included in ASTM D1143-81 as reapproved in 1987. This test method defines procedures for testing vertical or battered piles individually or in groups to determine the response of the pile or group to an axially applied static compressive load.

Additional methods for testing vertical or battered piles either individually or in groups include:

- the modified ASTM load test, which follows ASTM D1143-81 methods but may include modifications that must be approved by the Bridge Design Engineer;
- the ASTM D3966-90 test method, which covers procedures to determine the load-deflection relationships when the pile or pile group is subjected to lateral loads; and
- the ASTM D3689-90 test method, which covers procedures to determine the response of the pile or pile group to an axially applied static tensile load.

Before a pile load test is conducted, an economic comparison between the cost of the additional length of pile and the cost of the load test should be made. Generally, one pile load test will be conducted for each 100 piles required on a structure. Under unique circumstances, more tests may be specified. Refer to the *FHWA Manual on Design and Construction of Driven Pile Foundations*.

6.2.2.5 Pile Protection

Protection is needed for concrete piles where they are exposed. Protection should extend at least 5 feet [1.5 m] below stream bottom or ground surface. Steel piles should not be used for structures over water. Specify silane coating for concrete piles.

Soil conditions in some areas may require that Type II (sulfate-resistant) cement be used for concrete piles. The designer should check the soils report to determine if Type II cement or coatings are needed.

6.2.3 PILE-SUPPORTED FOOTINGS

Refer to the *FHWA Manual on Design and Construction of Driven Pile Foundations* for a discussion of pile-supported footings.

6.2.4 DRILLED SHAFT FOUNDATIONS

A drilled shaft is formed by boring an open cylindrical hole into the soil and subsequently filling the hole with concrete. Excavation is accomplished usually by a mobile drilling rig equipped with a large helical auger or a cylindrical drilling bucket. Once in place, a drilled shaft acts essentially like a driven pile, except that its behavior under load may differ because of the dissimilar geometries and installation techniques.

The following special features distinguish drilled shafts from other types of foundations:

- Unlike a displacement pile, the drilled shaft is installed in a drilled hole.
- Wet concrete is cast and cures directly against the soil forming the walls of the bore hole. Temporary casing may be necessary for stabilization of the open hole and may or may not be extracted.
- The installation method for drilled shafts is adapted to suit the subsurface conditions.

See Figure 6-18 for a sketch of a typical drilled shaft.

Other terminology commonly used to describe a drilled shaft includes: drilled pier, drilled caisson, and bored pile. Refer to the *FHWA Drilled Shaft Manual*.

6.2.4.1 Types of Drilled Shafts

The five categories of drilled shaft foundations are defined by their diverse methods of load transfer. Generally, the load-carrying capacity is obtained from load transfer to the soil from the shaft or the base or a combination of both, as described below.

1. Straight shaft, end-bearing drilled shaft. Load is transferred by base resistance only.
2. Straight shaft, side-wall-shear or friction drilled shaft. Load is transferred by shaft resistance only.
3. Straight shaft, side-wall-shear and end-bearing drilled shaft. Load is transferred by a combination of shaft and base resistance.
4. Belled or under-reamed drilled shaft. Load is transferred by the bell in end-bearing. Shaft resistance may be considered, depending on the dimensions of the drilled shaft and overburden material.
5. Straight or belled drilled shaft on hard soil or rock. Shaft resistance may be considered under some circumstances, with the approval of the Bridge Design Engineer.

6.2.4.2 Application of Drilled Shafts

The drilled shaft is usually employed as a deep foundation to support heavy loads or to minimize settlement. Because of the methods of construction, it is readily applied to soil above the water table, or soil that is nearly impermeable, and to profiles where rock or hard soil is overlaid by a weak stratum. With suitable construction techniques and equipment, the drilled shaft can be used in less favorable conditions. Casing or bentonite slurry can be employed to prevent caving or deformation of loose or permeable soils.

The methods of construction can be adapted to severely restricted conditions using specialized equipment. Often drilled shafts are used where piles cannot be driven due to physical overhead restrictions. Drilled shafts also have applications under certain environmentally sensitive conditions.

The geometry of the drilled shaft will be determined by the soil conditions and the performance requirements. If lateral forces have to be resisted, modifications to the structural stiffness must be made for the bending stress. The load capacity of drilled shafts is such that a single, large-diameter drilled shaft can take the place of a group of driven piles.

The flexibility of this type of foundation is such that axial and lateral loads can be resisted in a variety of soil conditions. The final decision as to whether drilled shafts are better applied to a foundation problem than driven piles must be based on the performance requirements and economic considerations. Refer to the *FHWA Drilled Shaft Manual*.

6.3 SEISMIC DESIGN AND RETROFITTING

All bridge structures in Delaware must be designed with consideration of seismic (earthquake) motion. Seismic design is not required for culverts. Each bridge will be designed considering the Seismic Performance Zone and the Acceleration Coefficient. Every bridge in Delaware must have an Importance Classification assigned.

The boundary between Seismic Performance Zone "1" and "2" is Delaware

S.R. 273. All bridges on, over or north of S.R. 273 should be designed using Zone "2", with an acceleration coefficient of 0.10 for calculations.

As bridges are rehabilitated, each must be evaluated for seismic considerations, and, with the approval of the Bridge Design Engineer, the necessary retrofitting construction will be included in the rehabilitation.

Refer to the *AASHTO Specifications* for specific criteria for seismic design.

6.4 ABUTMENT DESIGN

Abutments support the end spans of the bridge and retain the approach roadway embankment. The types of abutments used in Delaware are:

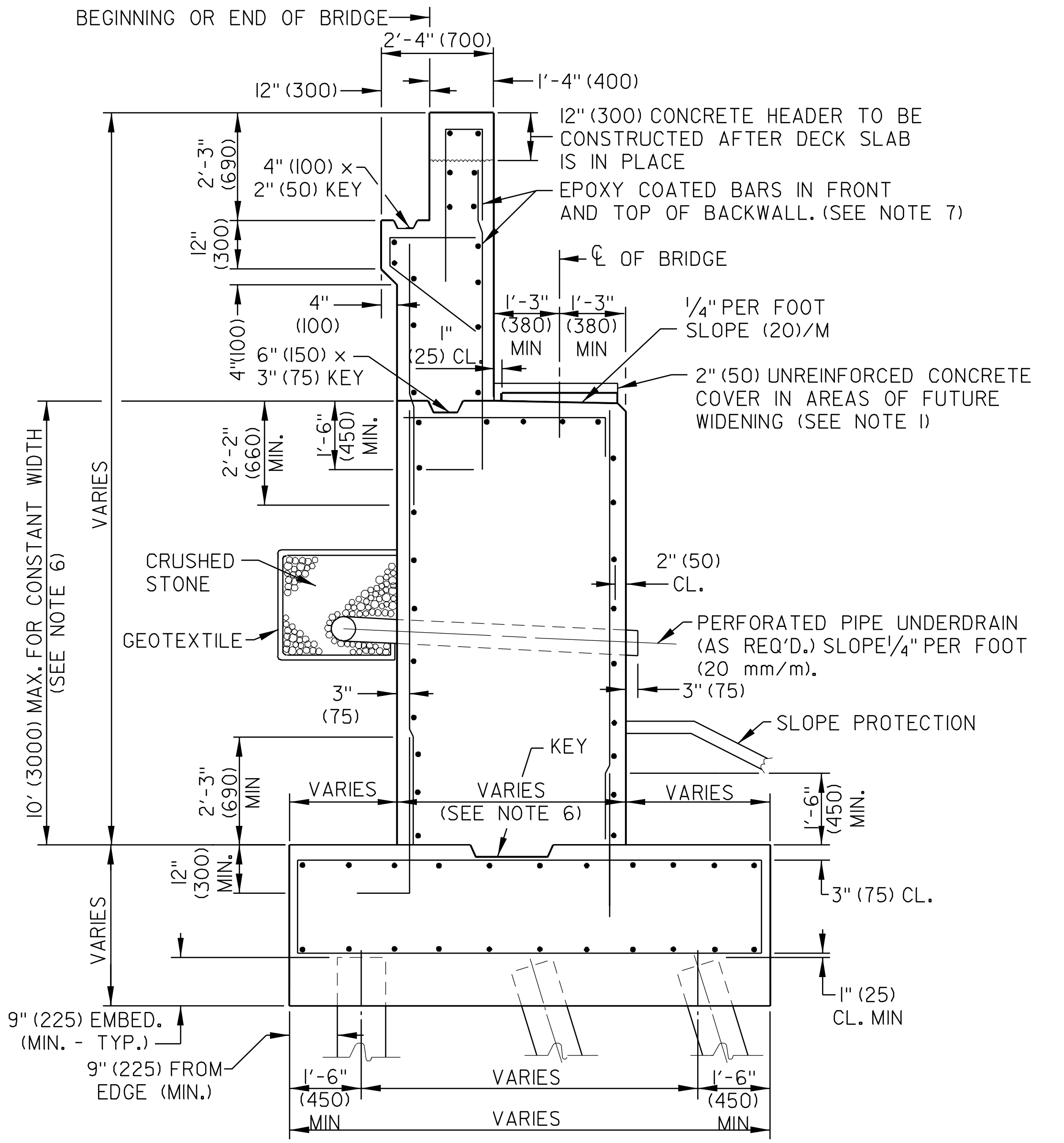
- cantilever (Figure 6-21);
- stub (Figure 6-22); and
- integral.

As noted in Section 6.2, the designer must evaluate the foundation conditions below the bottom of the footing. Where foundation conditions are acceptable, abutments on spread footings are permitted. Spread footings may be constructed on fills only with approval of the Bridge Design Engineer. Where the foundation soil cannot support the loads, piles are used to support the footing.

The designer must consider possible alternative construction sequences to assure that all loads applied to piles are considered in the design. Specifically, the placement of fill after the piles are driven can cause downdrag on the piles. Downdrag is considered a permanent load in the *AASHTO Specifications,* and shall be calculated in accordance with Section 3.11.8 of that document, applied to the appropriate load combinations. It is possible to develop negative skin friction in some soils, and the designer must consider auguring through these soils to preclude this condition. Battered piles should be avoided in negative skin friction situations because of the additional bending forces imposed on the piles. Downdrag can be reduced by specifying coating that portion of the pile subject to downdrag with bitumen.

The design compressive strengths of concrete (f'_c) are listed in Section 812 of the Delaware Standard Specifications. Specify Class A concrete (f'_c = 4,500 psi [30 MPa] at 28 days) for exposed abutments, stems, backwalls, and wingwalls and Class B concrete (f'_c = 3,000 psi [20 MPa]) for abutment and wingwall footings that are not exposed.

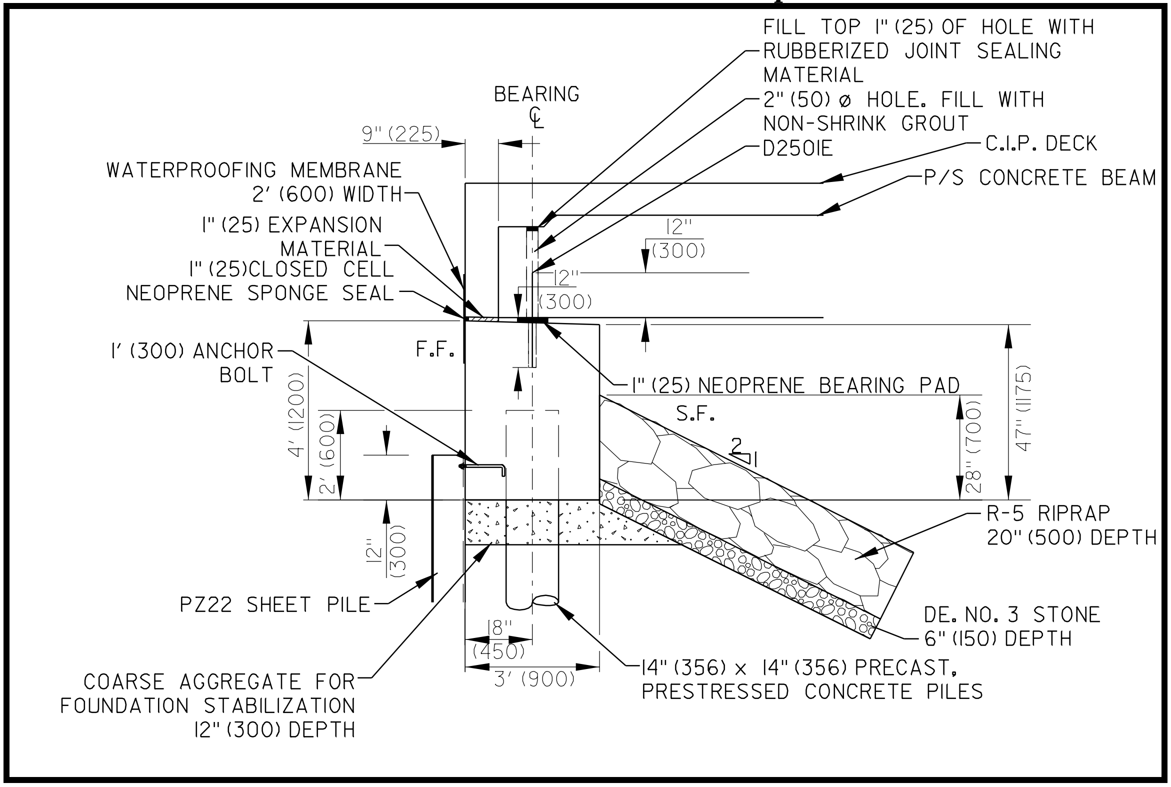
Figure 6-21
Cantilever Abutment Detail Example



SAMPLE SECTION
NTS

NOTE:
1. CONTINUOUS ABUTMENTS BE UTILIZED ON STRUCTURES WHERE FUTURE WIDENING INTO THE MEDIAN IS LIKELY.
2. PILE SIZE ARRANGEMENT (LAYOUT) AND TYPE, AS FOUNDATION CONDITIONS WARRANT.
3. MAXIMUM PILE BATTER IS 4:12.
4. REBAR MAT PATTERN OVER PILES SHALL BE SHOWN IN PLAIN VIEW SHOWING PATTERN.
5. REBAR SHOWN FOR GENERAL LAYOUT. ACTUAL SIZE AND LOCATION TO BE DESIGNED.
6. FOR HIGH WALL, BASE (STEM) OF WALL MAY BE STEPPED IN TO REDUCE CONCRETE REQUIRED.
7. UNCOATED BARS SHALL NOT COME IN CONTACT WITH COATED BARS.

Figure 6-22
Stub Abutment Detail Example



Reinforcing steel shall conform with AASHTO M31, Grade 60 [M31M, Grade 420]. The minimum cover for reinforcing steel is 2 inches [50 mm] for formed concrete. Where concrete is placed against soil, the minimum cover is 3 inches [75 mm].

6.4.1 CANTILEVER ABUTMENTS

Cantilever abutments are commonly used in Delaware. A sketch of a cantilever abutment is shown in Figure 6-21.

The breast wall provides for the reactions from the superstructure and also resists the thrust from the earth backfill. It is designed to resist this thrust as a retaining wall, cantilevered from the footing.

Wingwalls shall be carried down to the footing throughout their entire length. They may also be flared outward in plan view, depending upon the need for confinement of the earth fill. The structural action will depend upon the particular geometry. Cantilevered wingwalls will not be used because of the difficulty of compacting under the cantilevered portion of the wall.

The height of the cantilevered abutment should not exceed 25 feet [8 m]. One method to control abutment height is by lengthening the spans.

6.4.2 STUB ABUTMENTS

Stub abutments are used in situations where the need for retainment is minimal. A simplified sketch is shown in Figure 6-22.

Stub abutments are frequently built on pile foundations. In these instances, the bridge seat acts as a pile cap and must be sufficient to carry, by beam action, the loads from the bridge superstructure to the pile foundation.

Stub abutments are utilized in the majority of cases where a highway crosses another highway. A proprietary retaining wall may be placed in front of the abutment. See Section 6.6.1.4.

Where a highway crosses a stream, stub abutments may be used with steel sheeting placed behind the backwall to retain the approach fill material.

6.4.3 INTEGRAL ABUTMENTS

Integral abutments are another type of short abutment used where span lengths are short. The abutment and deck are poured monolithically, thus the name. The abutment normally encases the tops of the piles. This type of abutment is free to rotate because only a single row of plumb piles is used. Integral abutments may be used with the permission of the Bridge Design Engineer for structures with 30-degree skews or less.

6.4.4 DESIGN METHODOLOGY

A well-designed abutment provides safety against the possibility of overturning about the toe of the footing, against sliding on the footing base and against crushing of foundation material or overloading of piles. Abutments shall be proportioned in accordance with Section 11.5, Limit States and Resistance Factors, in the *AASHTO Specifications*.

6.4.5 DESIGN LOADS

The forces acting on an abutment are summarized in Section 3, Loads and Load Factors, in the *AASHTO Specifications*.

6.5 PIER DESIGN

There are multiple criteria and considerations to be evaluated in selecting the most economical and structurally appropriate type of pier to be designed. These include:

- separate or continuous footings;
- footing size;
- type of pier-column, solid shaft or hammer-head;
- number, spacing and size of columns;
- shaft dimensions; and
- cap size.

All of the forces that act on abutments also must be considered in the design of piers. In addition, stream, ice and drift forces must be considered. Refer to Section 3, Loads and Load Factors, in the *AASHTO Specifications*.

6.5.1 FRAME AND MULTI-COLUMN PIERS

Generally, one- and two-column piers are not to be considered due to the lack of redundancy. In certain situations, (i.e., very tall, very large columns) they may be viable.

The minimum pier column diameter is 2'-6" [750 mm] with 3'-0" [900 mm] preferred. Loading conditions may dictate a larger column.

Multiple-column piers are more economical in normal highway-over-highway construction, round columns being the most economical column type. Depending on the pier length, three or more columns are usually used.

6.5.1.1 Reinforcement

Care should be used in spacing vertical column bars to avoid excessive interference with the pier cap reinforcement. Double rows of column bars or large-diameter columns should be considered to alleviate this problem.

All spiral steel used for column reinforcing will be extended into the footing to the point of tangency of the vertical column reinforcement and into the cap a minimum of 6 inches [150 mm] to increase seismic resistance. Maintain the same pitch as in the column. See Figure 6-23 for details.

6.5.1.2 Construction Joints

If pier columns are over 30 feet [9 m] high, a construction joint should be placed at approximately mid-height.

6.5.1.3 Column Spacing

Columns should be spaced far enough apart to be appealing to the eye. The minimum center-to-center spacing is 15 feet [5 m].

6.5.1.4 Pier Caps

Pier caps should be proportionally sized to the columns. The minimum width of a cap is 2'-10" [865 mm] or the diameter of the column plus 4 inches [100 mm], whichever is greater.

Figure 6-24 provides specific dimensional relationships between elements for overpass structures.

Figure 6-23a
Seismic Reinforcement for Columns

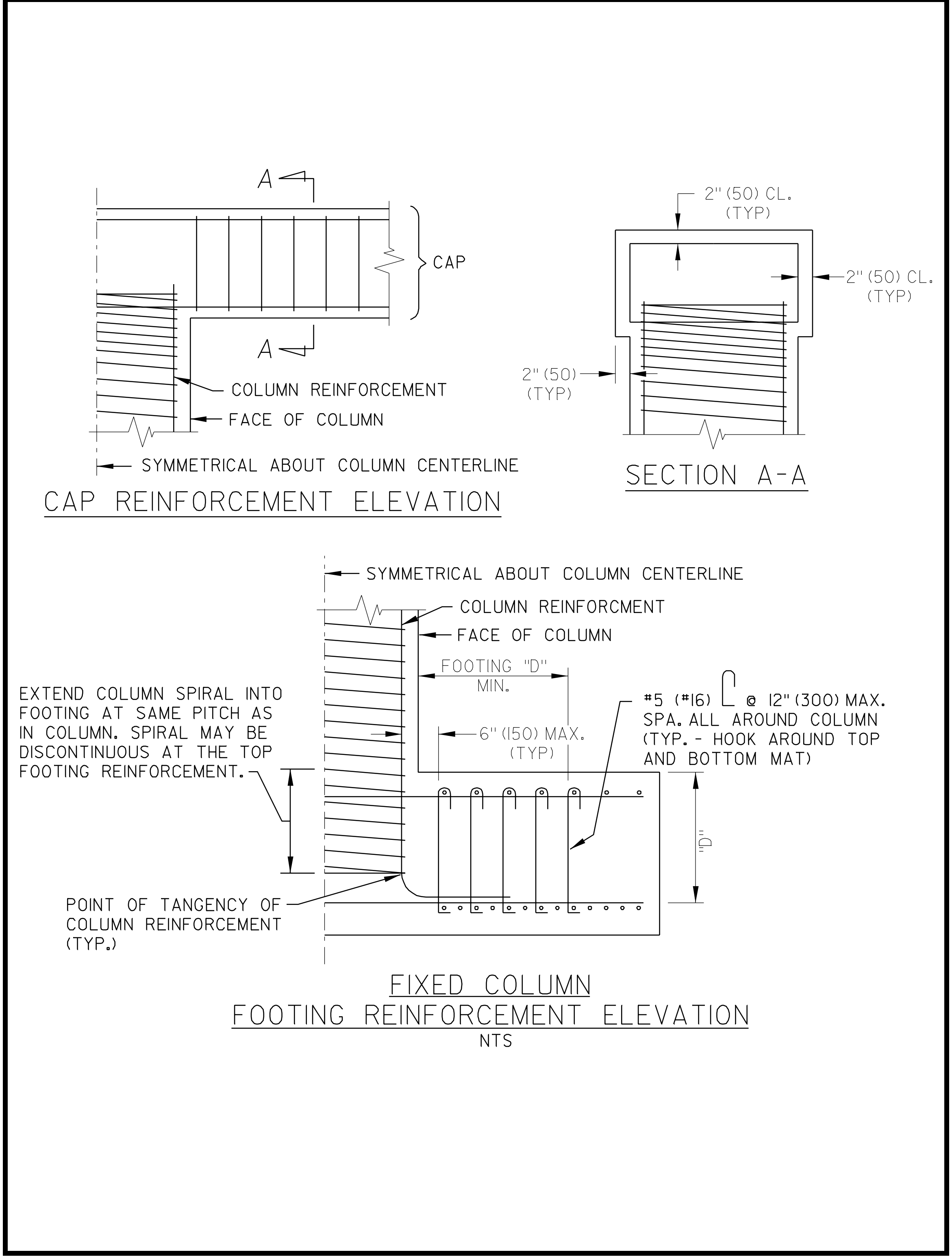


Figure 6-23b
Seismic Reinforcement for Columns

Notes to Designers:

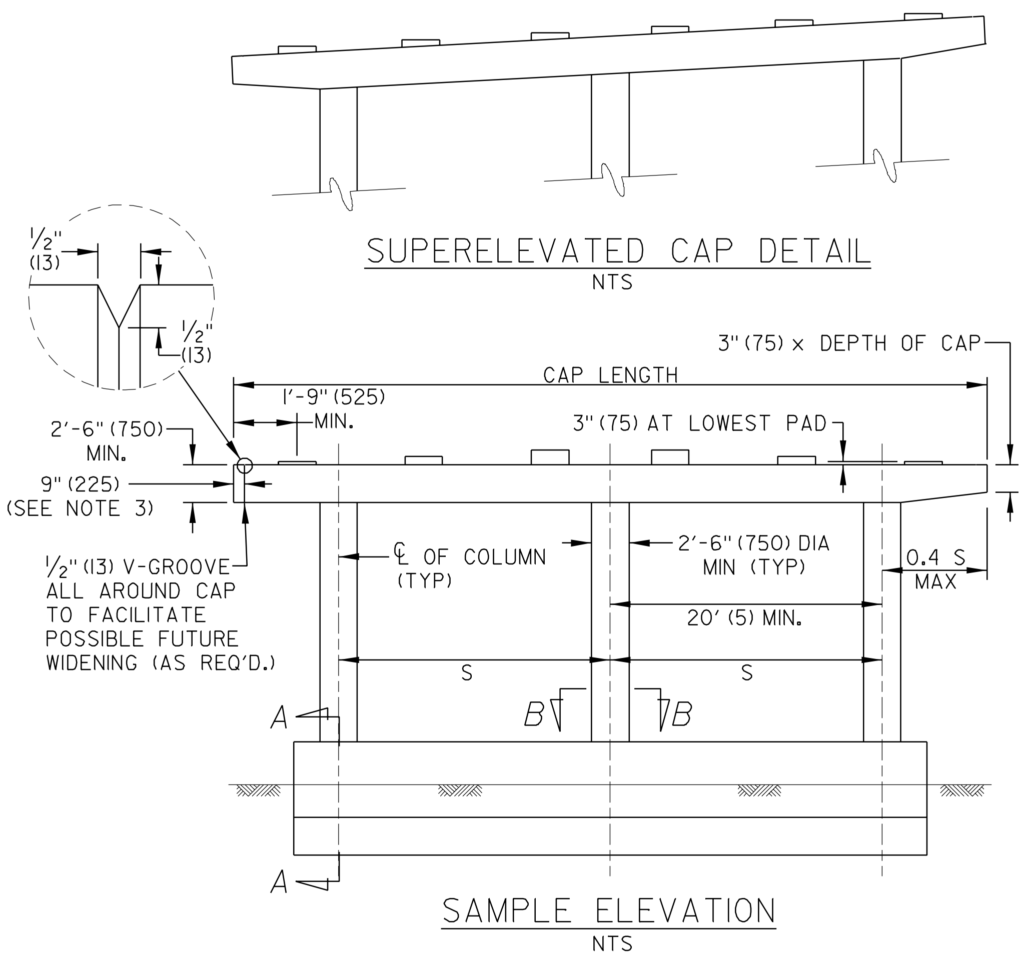
1. #5 [#16] bars @ 12 in [300 mm] spacing is minimum equivalent required.

2. The minimum top flexural reinforcement for footings shall be that required to resist loads which cause tension in the top fiber.

3. Locations for permissible discontinuities in spiral reinforcing must be shown on the plans.

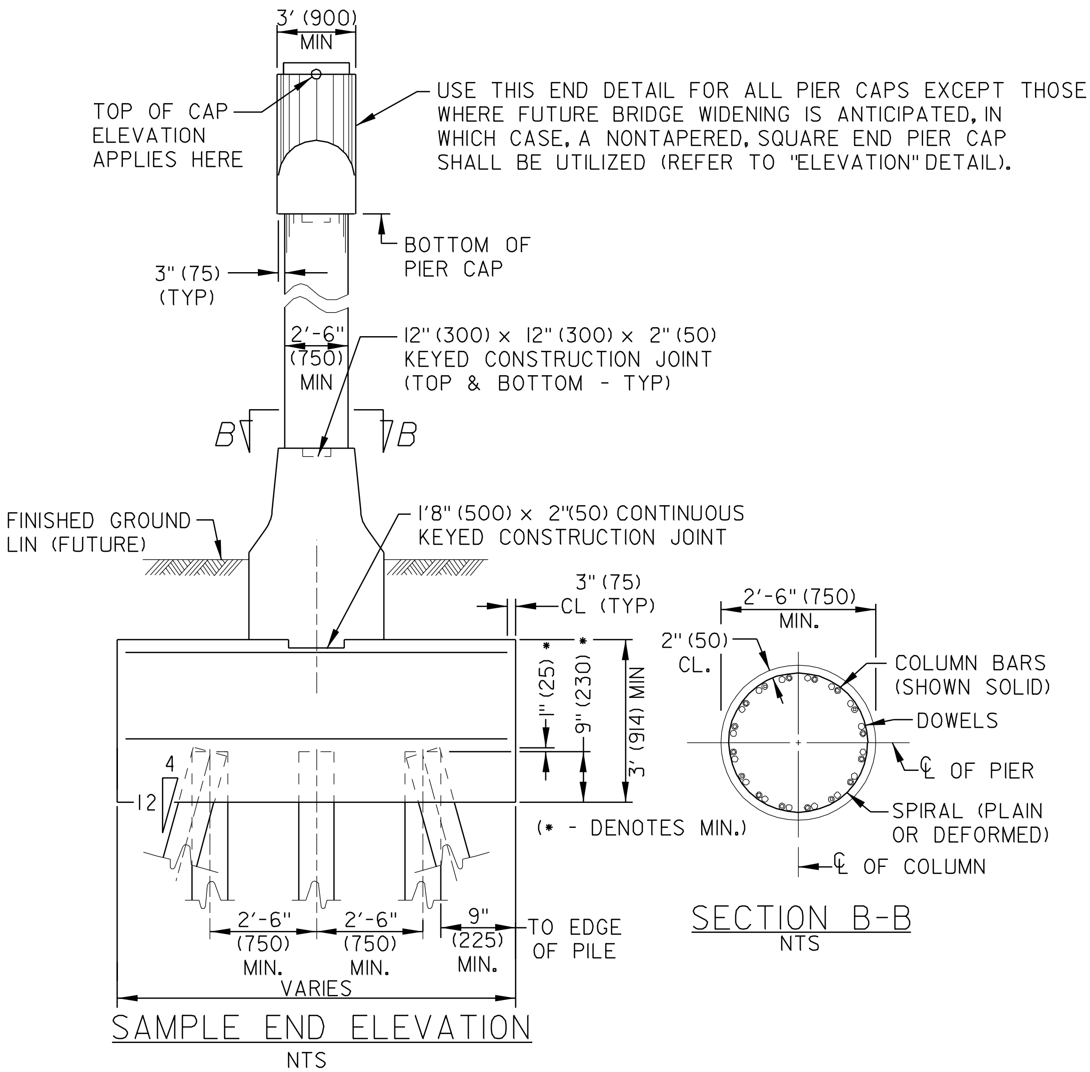
4. The thickness of the expansion joint filler should allow maximum column deflection without crushing the edge of the column concrete against the footing and should have a minimum thickness of 0.5 in [13 mm].

Figure 6-24a
Reinforced Concrete Pier Example



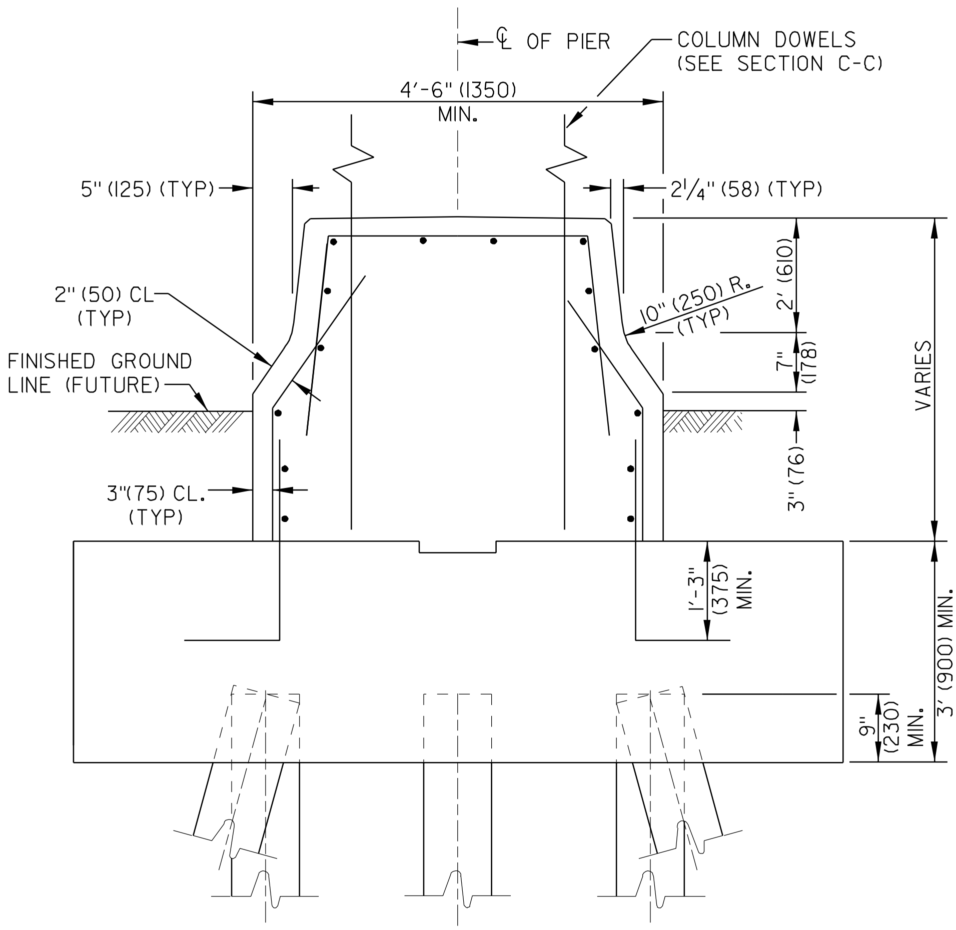
NOTE:
1. STANDARD IS FOR ROADWAY WIDTHS AND SKEW ANGLE REQUIRING A CAP LENGTH OF UP TO 60' (20), MEASURED ALONG CENTERLINE OF PIER.
2. WHEN BRIDGE SEAT ELEVATIONS ARE SUCH THAT THE HEIGHT OF ANY PAD BECOMES GREATER THAN 12" (300) AND THE SLOPING OF THE CAP CAN ELIMINATE OR ALLEVIATE THIS CONDITION, THEN THE CAP SHALL BE SLOPED AS INDICATED IN THE "SUPERELEVATED CAP DETAIL".
3. ALL MAIN BARS TO BE EXTENDED INTO THIS AREA SO THAT MECHANICAL EXTENSION CAN BE UTILIZED. DESIGNER MUST KEEP IN MIND, THIS MIGHT NECESSITATE MORE STEEL THAN REQUIRED FOR ORIGINAL DESIGN.
4. BOTH ENDS OF PIER CAP TO BE TAPERED EXCEPT WHERE FUTURE BRIDGE WIDENING IS ANTICIPATED, IN WHICH CASE, BOTH ENDS SHOULD BE SQUARE AND THE END TO BE WIDENED SHOULD BE DETAILED AS SHOWN ABOVE.
5. ALL DIMENSIONS ARE IN INCHES AND MILLIMETERS.

Figure 6-24b
Reinforced Concrete Pier Example



NOTE:
1. PILE SIZE, ARRANGEMENT (LAYOUT) AND TYPE, AS FOUNDATION CONDITIONS WARRANT.
2. BATTER PILES AS NECESSARY (MAXIMUM PILE BATTER IS 4:12).
3. REBAR MAT PATTERN OVER PILES SHALL BE SHOWN IN PLAN VIEW SHOWING PATTERN.
4. REBAR SHOWN FOR GENERAL LAYOUT, ACTUAL SIZE AND LOCATION TO BE DESIGNED.

Figure 6-24c
Reinforced Concrete Pier Example



SECTION A-A
NTS

NOTE:
1. REBAR SHOWN FOR GENERAL LAYOUT ONLY.
2. ALL BARS IN PEDESTAL SHALL BE EPOXY COATED.

6.5.2 SOLID OR HOLLOW SHAFT PIERS

Where space for large footings and multiple column piers is limited or columns are very high, solid or hollow shaft columns can be considered. Aesthetic treatment is preferred for massive concrete elements.

6.5.3 PILE BENTS

Pile bents have most recently proven to be the most economical type of pier. They are generally most suited for structures crossing rivers, of low- to mid-level clearance and multi-span structures.

Where piles are subject to wet and dry cyclic exposure, only concrete piles with pile protection are used. The protective coating is applied to the surface of precast-prestressed concrete piles after the pile is cast. Steel shell piles are not used in water because of durability and environmental impacts involving maintenance cleaning and painting.

Generally, precast-prestressed concrete piles are more economical than fluted steel shells or pipe piles. Precast-prestressed concrete piles are fabricated in one piece to a length defined by the designer. It is preferred that they are not field spliced. Where piles can be barged to the construction site, piles in excess of 100 feet [30 m] in length can be used. Where piles must be driven to an elevation lower than the bottom of the cap to achieve bearing, cap heights may be increased.

The minimum pile size is 18 inches [450 mm], either in diameter or square.

Point of Fixity. Piles are frequently extended above ground level to form frame support structures such as piers and bents. Their design is performed using standard structural design concepts. The principal problem in the design of these structures is the bending and buckling of the partially embedded piles. In evaluating possible buckling of a partially embedded pile and in performing frame analyses, it is necessary to estimate the lower condition of fixity. The term fixity indicates restraint against rotation only. See Figure 6-25.

The effective length equals KH for analysis of allowable axial loads. See Figure 6-26 for the recommended design values for K. These values are for a pile assumed to be fixed at the bottom.

The depth of the point of fixity can be computed for uniform coarse- and fine-grained soils by the following equations. If the soil profile is not uniform, a more detailed analysis is required. Software, such as COM624P or STAAD, may be used for the analysis. Refer to Chapter 12.

Where a soft layer overlays a hard layer, fixity is difficult to attain. The designer may use a hinged design with approval of the Bridge Design Engineer. The stability of the structure must be carefully investigated.

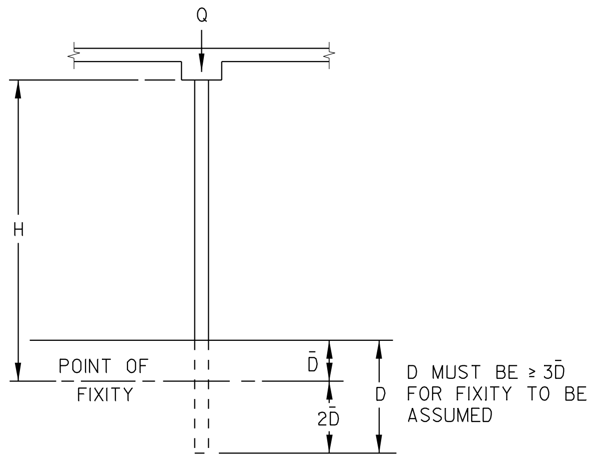
6.6 RETAINING WALL DESIGN

Retaining walls are designed to withstand lateral earth and water pressures including live and dead load surcharges, the weight of the wall, temperature and shrinkage effects, and earthquake loads in accordance with Section 11.5, Limit States and Resistance Factors, in the *AASHTO Specifications*.

Passive pressure resistance to sliding from soil in front of the wall will not be considered without permission from the Bridge Design Engineer.

See Chapter 12 for available computer software for the design of retaining walls.

Figure 6-25
Point of Fixity



Note: This figure was adapted from Table C1.8.1, *Specification for the Design, Fabrication and Erection of Structural Steel for Buildings*, November 1, 1978, with permission of the American Institute of Steel Construction.

Figure 6-26
Recommended K Values

DIRECTION	LONGITUDINAL		TRANSVERSE	
LOCATION TYPE	2 ROWS BATTERED	SINGLE ROW	BATTERED END	PLUMB PILE
RECOMMENDED DESIGN K VALUE	0.65	1.6	0.65	1.2

Note: Adapted from "Lateral Resistance of Piles in Cohesionless Soils," by Bengt B. Broms, Volume 90 No. SM3, Part I, *Journal of the Soil Mechanics and Foundation Division*, Proceedings of the American Society of Civil Engineers, May 1964, and reproduced by permission of ASCE.

6.6.1 TYPES

The following are some commonly used types of retaining wall structures available for the designer to consider in a specific design:

- Post and Plank Walls
- Sheet Pile Walls
- Reinforced Concrete Walls
- Proprietary Retaining Walls

6.6.1.1 Post and Plank Walls

Post and plank walls consist of steel H-piles driven or augured at designated spacings. The piles may be tied back using drilled or grouted type anchors. The spaces between the piles are spanned with structural elements such as wood, reinforced concrete lagging, precast or cast-in-place concrete panels or steel members, to retain the soil.

6.6.1.2 Sheet Pile Walls

Sheet piling walls may be either cantilever or anchored design. Sheet piling is driven in a continuous line to form a wall. In cantilever design, fill is then placed and compacted behind the wall. Cantilever walls are generally limited to 15 feet [5 m] in height. In anchored design, deadmen or piles are then constructed, and the sheeting wall is anchored to them using tie rods.

6.6.1.2.1 Steel Sheet Piles

Steel sheet piles are used for both temporary and permanent construction. Both tied-back and cantilever designs are allowed. The contractor is responsible for the design of temporary structures, with approval of the designs by the Department.

Where steel sheeting is used as permanent construction, a coating is required. Where a cap is required, a concrete cap is preferred.

A690 sheet piles should be used in marine environments such as tidal areas.

A709 Grade 36 [A709M Grade 250] and A709 Grade 50 [A709M Grade 345] sheet piles are used in non-marine environments. Both types are always coated.

Steel sheet pile retaining walls are used as sea walls and for similar types of shore protection such as flood walls, levees, and dike walls used to reclaim lowlands.

In no situation will an abutment be constructed using driven steel sheet piling as support for the structural loads.

Designers should refer to the AISC *Sheet Piling Design Manual*.

A computer program, CWALSHT, is available for design and analysis of steel pile walls by classical methods. The program was developed by the Corps of Engineers. See Chapter 12 of this manual for details.

6.6.1.2.2 Concrete Sheet Piles

Concrete sheet piles are precast, prestressed concrete members designed to carry vertical loads and lateral earth pressure which act as abutments. These members are connected by a keyed vertical joint between two adjacent sheets. Geotextile fabric or suitable joint sealer is used to prevent loss of backfill material through these joints. The sheets are driven to ultimate bearing capacity using water jets except that the last 12 to 15 feet [3.6 to 4.6 m] are driven using suitable hammer. The use of concrete sheet piles is permissible in sandy soils only, with approval from the Bridge Engineer.

6.6.1.3 Reinforced Concrete Walls

Reinforced concrete walls are constructed using cast-in-place or precast concrete elements. They may be constructed on spread footings or founded on piles. They derive their capacity through combinations of dead weight and structural resistance.

6.6.1.4 Proprietary Retaining Walls

Proprietary retaining walls are patented systems for retaining soil. Two types of systems used in Delaware are gravity and mechanically stabilized. Gravity walls generally use interlocking, soil-filled reinforced concrete bins or modular blocks to resist earth and water pressures; they depend on dead weight for their capacity. Mechanically stabilized walls use metallic or polymeric tensile reinforcement in the soil mass and modular precast concrete panels to retain the soil.

In locations where retaining walls are needed to reduce span lengths or facilitate construction, proprietary walls may be considered. Consideration of economics, location, construction requirements, and aesthetics should be included in the evaluation. These walls have proven to be very economical to build, especially for long abutments. They should also be considered when constructing a dual highway over secondary side roads. This type of construction can also reduce span lengths, thus saving on superstructure construction costs. Proprietary retaining walls can be economical where high wall heights are dictated by field conditions.

Locations where proprietary walls should be considered are based on the following requirements:

- readily available required backfill material,
- available site construction working area, and
- insufficient right-of-way.

All abutments constructed behind proprietary retaining walls will be founded on piles. Spread footings will not be permitted. Proprietary retaining walls are used to retain earth and do not carry vertical structure loads. Abutments are still used to support all longitudinal loads.

Each design location must be evaluated based on the advantages and disadvantages of the specific construction being considered. This is particularly important when a mechanically stabilized wall is being considered for a roadway crossing over a waterway. Close consideration must be given to long-term stability, stream flow, and storm flows. Positive erosion control in addition to geotechnical fabric is needed. These walls should not be used in tidal areas or other locations where water might reach the wall.

When proprietary retaining walls are included in a project, special provisions must be included in the contract documents to guide the suppliers. The wall suppliers provide all required engineered designs of the structural wall. Suppliers' designs are included in the plans. The contractor selects a supplier's design and submits a bid accordingly.

Proprietary walls that are currently approved for use in Delaware are listed below. Other walls may be approved by the Bridge Design Engineer.

- Double Wall™ (gravity wall)
- VSL™ (mechanically stabilized)
- Reinforced Earth™ (mechanically stabilized)

6.7 APPROACH ROADWAY EMBANKMENT

In many cases, approach roadways are constructed on embankments. When embankments are constructed over soft soils, the designer shall consider the potential for settlement and slope stability problems. Embankment slopes also require protection from erosion.

6.7.1 SETTLEMENT

Often the major design consideration when faced with a settlement problem is the time for the settlement to occur. Low-permeability clays and silt-clays can take a long time to consolidate because the water must be squeezed out before the consolidation is complete.

The two most common methods of accelerating settlement are:

- applying a surcharge; and/or
- the use of sand or wick drains in the subsoil.

Surcharges involve building the embankment above the final grade elevation and allowing it to remain for a period of time, typically 3 to 12 months. The length of the waiting period can be estimated from consolidation test data. The actual settlement occurring is monitored with geotechnical instrumentation. When the settlement with the surcharge equals the settlement originally estimated (without the surcharge) and the rate of settlement is reduced to an acceptable level, the surcharge may be removed. (Refer to Figure 6-27.) The designer may also use the surcharge load or duration to achieve secondary settlement. The designer should specify the following:

- the estimated time to achieve settlement,
- the total settlement,
- the desired rate of settlement, and
- the rate of placement of the surcharge.

The stability of the embankment must be checked against slope failure to ensure that an adequate factor of safety exists to permit placement of the surcharge load. It may be necessary to specify the rate of placement of the surcharge or stage surcharge construction to control the pore pressure in the soils and maintain stable slopes. Piezometers and settlement platforms are installed at various locations in the fill to monitor pore pressures during the loading and consolidation phases. Locations for these will be established by M&R.

Some highly plastic clays of extremely low permeability can take many years for settlement to be completed. When surcharging alone is not effective in reducing settlement time sufficiently, wick drains can be used to accelerate the settlement; because of the increased cost, wick drains should be used only when this condition exists. These drains accelerate the settlement process by shortening the drainage path for the water to escape from the soil, as shown in Figure 6-28. When wick drains are needed, the designer should request that M&R prepare the design.

Figure 6-27
Surcharge Settlement vs. Time

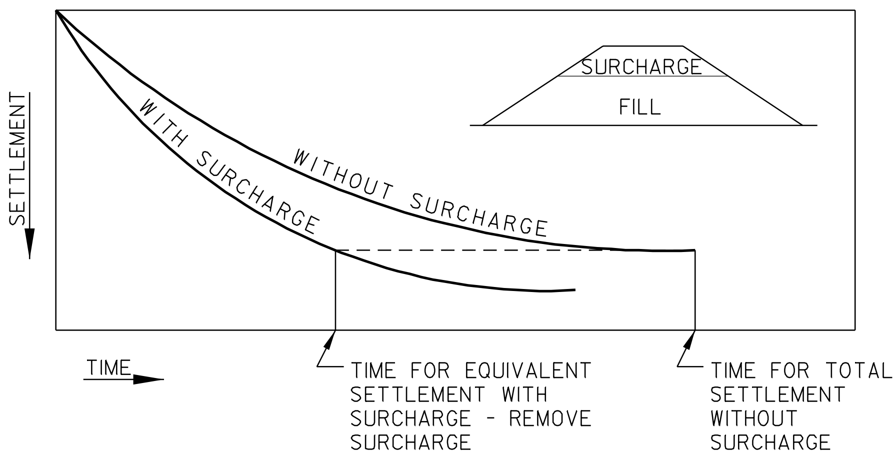
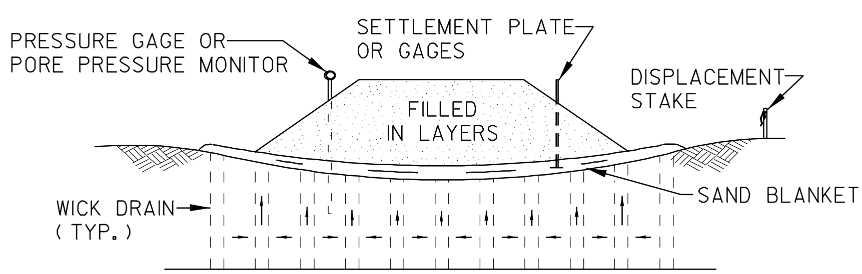


Figure 6-28
Wick Drains



6.7.2 SLOPE STABILITY

Designers must be aware of potential slope stability problems and request that M&R check potential problem areas. There are three major types of stability problems that should be considered in the design of approach embankments over weak foundation soils: circular arc failure, sliding block failure, and lateral squeeze of foundation soils. These are shown in Figures 6-29 through 6-31.

Computer software PCSTABL6 is available in the Bridge Section and M&R to analyze slopes and bridge approach embankments for stability. Refer to Chapter 12. The applications and analysis methods are summarized in Figure 6-32.

End slopes beneath bridge abutments, major retaining walls and other locations where slope failure would result in significant damage shall be designed in accordance with Section 10.5.2, Service Limit States, in the *AASHTO Specifications*.

Figure 6-29
Circular Arc Failure

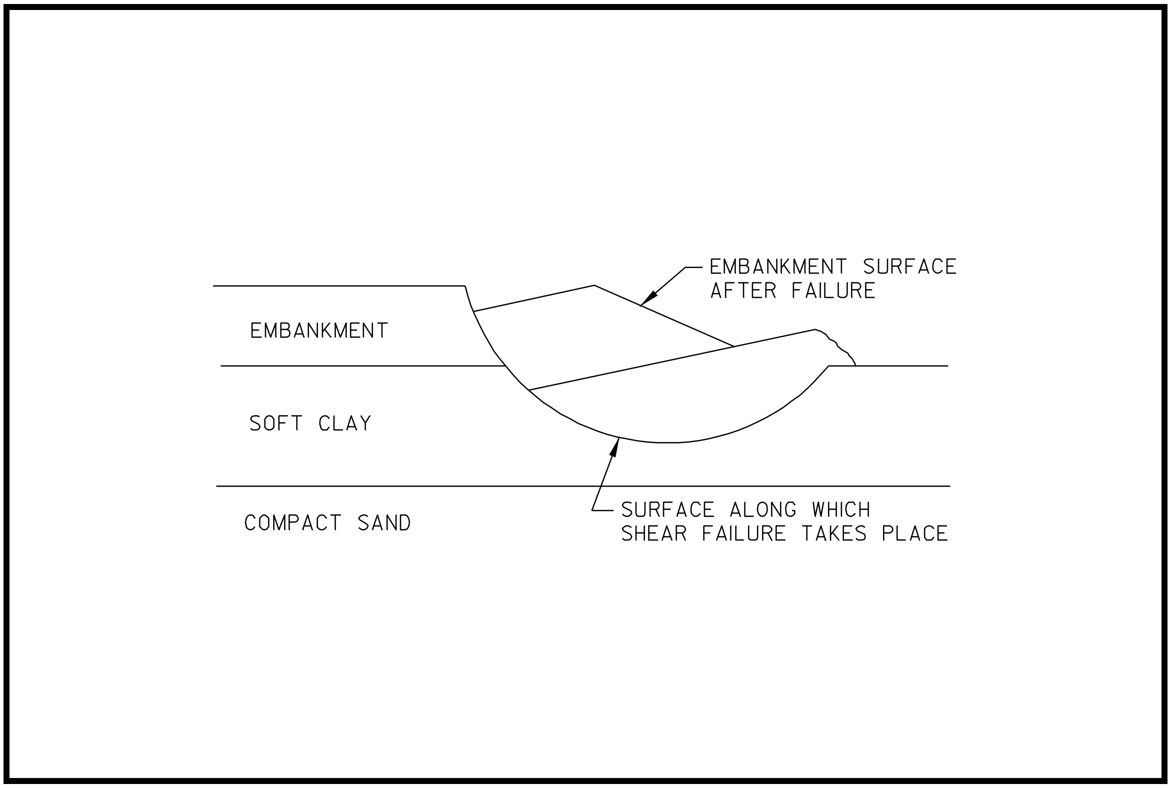


Figure 6-30
Sliding Block Failure

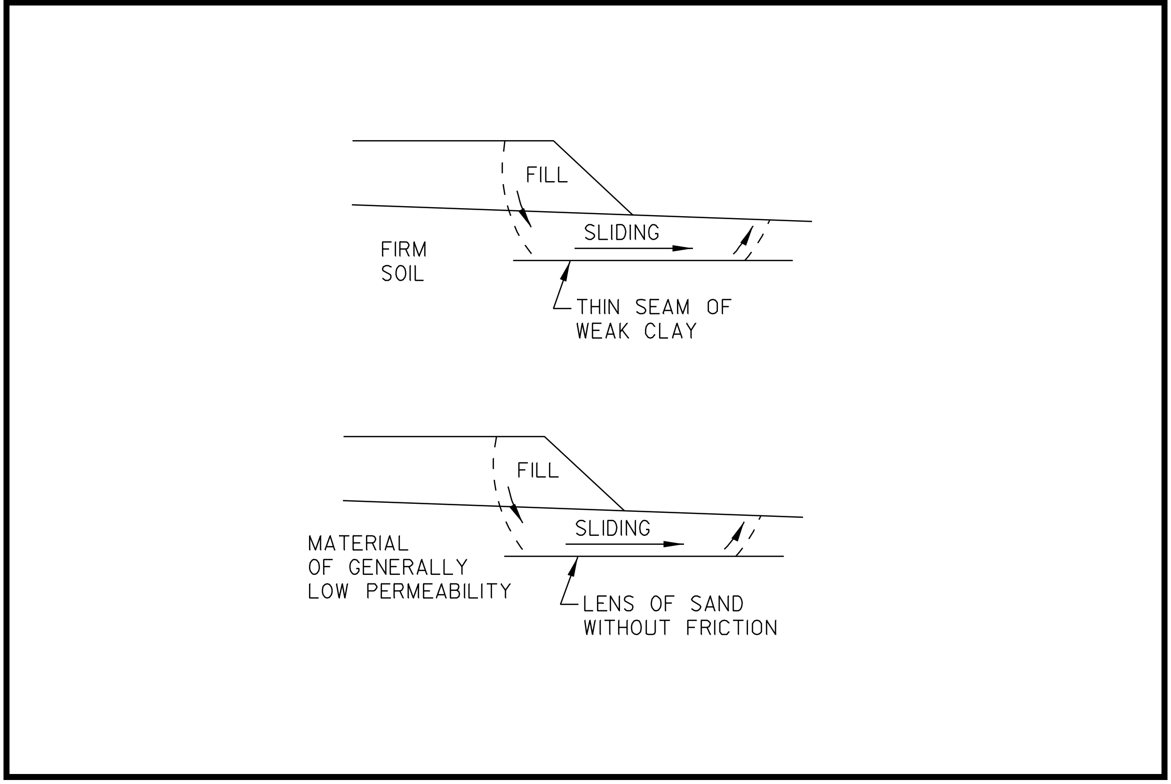


Figure 6-31
Lateral Squeeze of Foundation Soil

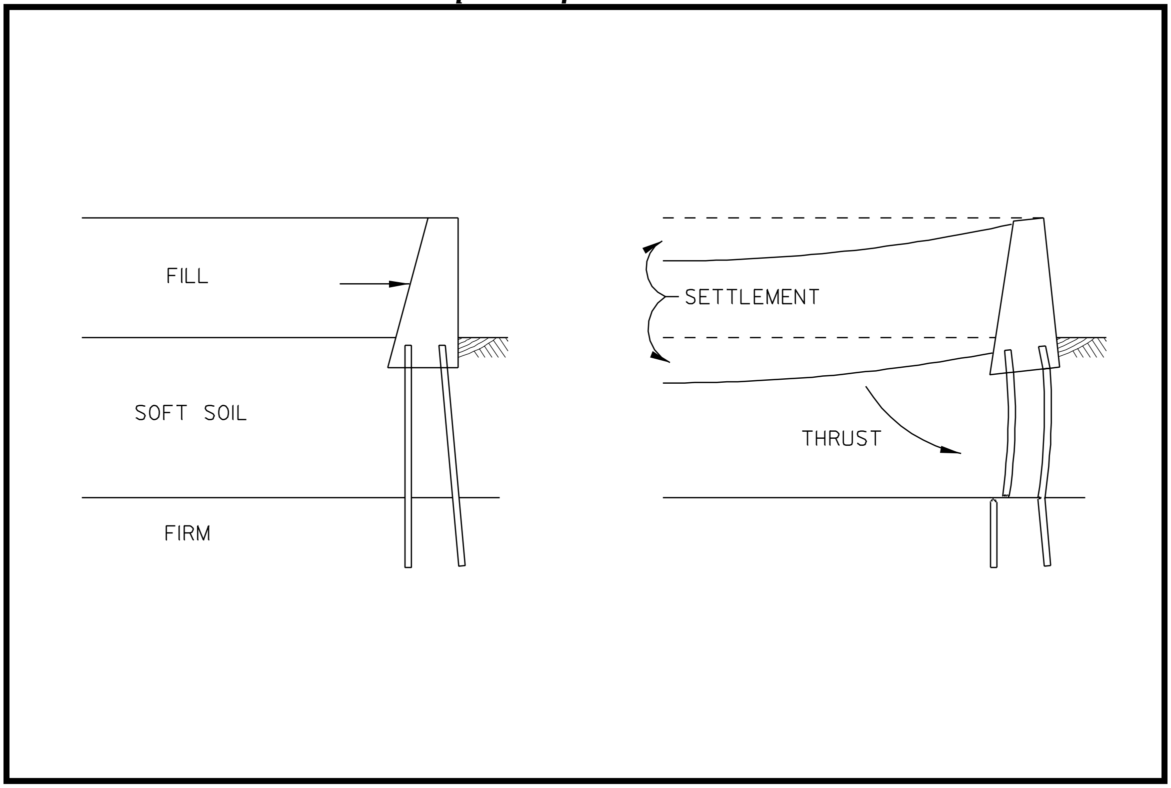


Figure 6-32
Slope Stability Software Applications

Software	Method	Application
FHWA PCSTABL6	1. Simplified Bishop	Circular shaped failure surface
	2. Janbu	General shaped failure surface (non-circular)
	3. Spencer method of slices	Any type of surface failure
	4. Sliding block	Weak layer or strong layer

6.7.3 SLOPE AND BANK PROTECTION

Slope protection or bank protection is placed for several reasons. The primary purpose is to protect the embankment or slope from erosion. The primary cause of erosion is stream action, but it may also be caused by stormwater runoff.

6.7.3.1 General

To design bank protection properly, knowledge of how the bank fails is important. The ways in which banks fail are:

- erosion of soil particles by river currents, waves or scour;
- bank sloughing caused by internal hydrostatic pressure in the bank materials;
- slip-circle failure undermining the toe; and
- liquefaction.

The goal is to protect the embankment and to define the flow area through the stream crossing. Where channels are expected to remain stable or where the embankment (bridge berm) is set back from the stream bank with only minor constriction at lower frequencies, it is not necessary to protect the bank, but consideration should be given to protecting the bridge berm at the design frequency flow. At sites where the embankment coincides with the stream bank or is practically an extension thereof, protection of both the stream bank and embankment is recommended.

Where the abutments of dual structures are separated by at least 15 feet [4.5 m] of clear space, the stone slope protection need not be continuous between the abutments. This will be decided on a case-by-case basis.

In addition to bank protection, each substructure unit must be designed (or protected) following the latest scour considerations. It is not economically feasible to construct all bridges with absolute invulnerability from scour damage. Every bridge must be evaluated to determine the prudent scour prevention measures to be taken. The problems associated with estimating scour and providing cost-effective and safe designs are being addressed in current research programs. Currently, there are guidelines and publications available for designing scour protection and structures which resist scour. Refer to Chapter 3 of this manual. Also refer to *HEC-18*, *Evaluating Scour at Bridges*; *HEC-20, Stream Stability at Highway Structures*; *HEC-23*, *Bridge Scour and Stream Instability Countermeasures;* and the most recent research for state-of-the-art guidance.

6.7.3.2 Geosynthetics

Geosynthetics are permeable and nonpermeable polymer textile materials used in conjunction with soil and rock to provide one or more of the following functions:

- barrier;
- separation;
- filtration;
- drainage; and
- reinforcement.

Geosynthetics have been successfully used in the following highway applications:

- drainage;
- temporary and permanent erosion control;
- temporary and permanent pavements;
- embankments over weak foundations; and
- reinforced slopes and walls.

Design of geosynthetics is required for each application. Design and construction monitoring guidelines are in the publication *FHWA-HI-90-001*, *Geotextile Design and Construction Guidelines*.

6.7.3.3 Rock Riprap

The most common method of bank or slope protection is rock riprap. The sides of the bank or embankment are lined with large rocks to prevent erosion along the bank and at the toe. Rock riprap protection has advantages in that it is flexible and local damage is easily repaired. Appearance of the rock riprap is natural, and, in time, vegetation will grow between the rocks. Also, wave run-up on rock slopes is usually less than for other types of protection. Construction must be accomplished in a prescribed manner to assure proper behavior. The following factors should be considered in the design of rock riprap protection:

- the durability and density of the rock;
- the magnitude and direction of stream velocity;
- the angle of the side slopes;
- the size of the rock; and
- the shape and angularity of the rock.

Filter blankets are used as reverse filters to prevent piping damage to the riprap caused by movement of small particles up through the larger stone as a result of decreased hydrostatic pressure from flowing water. Stone bank protection should terminate with a buried toe.

Design guides for estimating rock size for channel and stream bank protection are included in Chapter 3 of this manual and *HEC-23, Bridge Scour and Stream Instability Countermeasures*.

Specify a minimum 1'-6" [450 mm] thick blanket for embankment protection and 2'-0" [600 mm] thick for slope protection along stream banks and for streambeds. The specifications for riprap are provided in

Section 712 of the *Standard Specifications*. Where unusual problems are anticipated or the adequacy of ordinary practice is uncertain, a complete detailed design of the riprap gradation and filter blanket is recommended. Typical riprap details and examples of riprap installations are found in Chapter Three.

See Chapter Two for more information.

Check with Environmental Studies if topsoiling and seeding is needed.

6.7.4 MISCELLANEOUS

Other methods of bank protection such as articulated concrete mattresses, gabions, concrete filled "bags" or other products may be considered on a case-by-case basis.

6.8 ARCHITECTURAL TREATMENTS

Architectural treatments are used to improve the aesthetics of bridges. Because of the extra cost, such treatments are warranted only at selected locations. Treatments include:

- brick facing;
- stone facing;
- exposed aggregate; and
- formliners.

Formliners are used on structures such as overpasses where a large part of the structure is visible. Formliners simulating various textures and treatments are available. They have been used to simulate stone and brick and can be considered on a case-by-case basis. They provide architectural treatment at lower cost than other types of treatments.

Generally, vandalism in the form of graffiti on bridge substructures is not common. Each bridge will be reviewed to determine its potential as a target for graffiti vandalism, and, if it is needed, an anti-graffiti coating will be specified.

6.9 SUBSTRUCTURE DRAINAGE

Any water that accumulates behind abutment back walls and retaining walls must be drained to prevent settlement of the embankment or failure of the wall. This is accomplished through footing drains, weep holes, and geosynthetic drains. Granular backfill behind the walls is essential to carry the water to footing drains and weep holes.

Footing drains are preferred over weep holes to drain walls that are visible to the public. A perforated drain pipe is installed behind the footing with outlets located to minimize aesthetic impacts. Weep holes may be used in walls that are not generally visible to the public, such as in back walls for stream crossings. Additional drainage for perched abutments where the granular backfill material extends below the abutment is not required. Geosynthetic drains provide drainage of the fill immediately behind the walls without the necessity of placing stone backfill. These drains are available in various thicknesses and capacities.