INDIAN RIVER INLET BRIDGE

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GEOTECHNICAL SUMMARY REPORT

REPLACEMENT OF 3-156, SR 1 OVER THE INDIAN RIVER INLET

DelDOT PROJECT NO. 1204

MACTEC PROJECT NO. 3530-03-1245

APRIL 26, 2005

Prepared By

MACTEC

For

Figg Engineering Group

In Cooperation With

DelDOT

MACTEC

April 26, 2005

Mr. W. Denney Pate, P.E.
Figg Bridge Engineers, Inc.
424 North Calhoun Street
Tallahassee, Florida 32301

SUBJECT: **Geotechnical Summary Report**
Indian River Inlet Bridge
Sussex County, Delaware
MACTEC Project No. 3530-03-1245.04

Dear Mr. Pate:

MACTEC Engineering and Consulting, Inc. is pleased to present this *Geotechnical Summary Report* for the Indian River Inlet Bridge project. This report presents a brief review of the project information provided to us, a short discussion of the general subsurface conditions, and a summary of our design and construction recommendations relevant to the current state of design (100%) for the project.

We have enjoyed working with you on this stage of the project and look forward to continuing to assist you on the remainder of the project. If you have any questions concerning this report, please do not hesitate to contact us.

Sincerely,

MACTEC ENGINEERING AND CONSULTING, INC.

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GEOTECHNICAL SUMMARY REPORT

INDIAN RIVER INLET BRIDGE

SUSSEX COUNTY, DELAWARE

DelDOT PROJECT NO. 1204

Prepared for:

FIGG BRIDGE ENGINEERS, INC.
Tallahassee, Florida

In Conjunction With:
RUMMEL, KLEPPER & KAHL, LLP
Dover, Delaware

MACTEC PROJECT NO. 3530-03-1245.04
April 26, 2005

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1.0 INTRODUCTION

1.1 GENERAL

The Indian River Inlet Bridge replacement project consists of the design and construction of a replacement structure for the existing State Route 1 (SR1) Bridge over the Indian River Inlet located in Sussex County, Delaware. Several geotechnical reports have previously been issued for this project. These reports are listed below.

- *Site Characterization and Preliminary Geotechnical Study*; Prepared by MACTEC and dated September 26, 2003.
- *Final Geotechnical Roadway Report*; Prepared by MACTEC and dated December 24, 2003.
- *Final Geotechnical Bridge Substructure Report*; Prepared by MACTEC and dated December 24, 2003.
- *Supplemental Investigative Foundations Report*; Prepared by Figg Bridge Engineers, Inc. and dated March 19, 2004.
- *Arch Base Foundation Recommendations*; Prepared by Figg Bridge Engineers, Inc. and dated April 27, 2004.
- *Abutment Foundation Recommendations*; Prepared by Figg Bridge Engineers, Inc. and dated June 23, 2004.
- *Summary of Design Soil Parameters*; Prepared by MACTEC and dated September 30, 2004.

These reports present geotechnical recommendations relevant to the design requirements of the project at the time each report was issued. However, as the project evolved, design requirements changed; significantly in some cases. Therefore the Delaware Department of Transportation (DelDOT) has requested that MACTEC prepare this report to summarize our geotechnical recommendations as they relate to the current state of design (100%) for the project.

It is our understanding that this report is intended only to briefly describe our design methods and summarize our current recommendations as they relate to the construction of the roadway embankments and bridge substructures for the subject project. The evaluations and recommendations developed for this report are based on project information supplied by Figg Bridge Engineers, Inc. (FIGG), and Rummel, Klepper, & Kahl, LLP. (RK&K). Additional information was obtained from DelDOT, including various as-built plans and boring logs for the existing bridge over the inlet. This project information, including reviews of area site and geologic conditions, results of our subsurface exploration and laboratory testing programs, estimated

subsurface profiles, and summaries of engineering parameters for typical subsurface strata are contained in the reports listed above. A brief summary of the general stratigraphy of the on site soils is given in Section 1.4 below.

1.2 SCOPE OF SERVICES

The preparation of this report is not included in MACTEC's original scope of services; however the scope of services associated with previous stages of this project is described in detail in MACTEC's recommended scope of work dated August 22, 2003. The purpose of the studies summarized herein was to utilize existing subsurface information, provide detailed information concerning the subsurface materials, and provide geotechnical design and construction recommendations for use in the design of bridge embankments and substructures. In order to provide the necessary information to perform these tasks, a total of forty standard penetration test (SPT) soil test borings and twenty piezocone penetration test (CPTu) soundings were performed at selected locations along the proposed alignment including eight soil test borings and two CPTu soundings in the vicinity of the planned bridge piers and abutments. Details of the exploration program including presentation of the data and interpretations are contained in the previously mentioned geotechnical reports.

In general, the borings and soundings were located in areas of geotechnical interest identified from review of existing subsurface data, preliminary plans, and information provided by FIGG. Our laboratory testing program was established to characterize the physical and chemical parameters of the subsurface soils. Our in-situ testing program consisting of pressuremeter tests, field vane shear tests, seismic shear wave velocity tests, and pore-pressure dissipation tests was implemented to aid in the characterization of the engineering parameters of the subsurface soils. The results of the field exploration and laboratory testing programs for both the bridge and roadway portions of the project were used in developing the recommendations contained in this report.

This report discusses our analyses which have been completed based on the collected data, presents our findings and evaluations, and generally includes the following:

- Geotechnical evaluations and recommendations for embankment construction including settlement estimates, global stability, lateral earth pressures, and design and construction of retaining walls.
- General recommendations regarding pavement design, including design CBR values, underdrain recommendations, and evaluations of ground-water effects on construction and pavement performance.
- Recommendations regarding compacted fills, including an evaluation of the suitability of on-site soils for use as fill material.
- Recommendations for areas requiring ground improvement, including discussion of performance of recommended ground improvement methods.
- Geotechnical recommendations for bridge pier and abutment foundations, including recommended foundation types, estimated bearing elevations, allowable axial compressive and uplift capacities, lateral loading design parameters, settlement estimates, installation, inspection, and load testing.
- Geotechnical recommendations for bridge abutment walls, including recommended design earth pressure coefficients.
- Recommendations regarding foundation construction, including monitoring and mitigation of construction induced vibrations on the adjacent existing bridge.

1.3 AUTHORIZATION

Our services were provided in general accordance with the Agreement for Subconsulting Services between MACTEC and FIGG dated June 18, 2003. The scope of services developed by MACTEC for the Final Geotechnical Study phase of this project was outlined in our memorandum dated August 22, 2003.

1.4 GENERAL STRATIGRAPHIC PROFILE

The soil stratigraphy on either side of the inlet in the vicinity of the proposed bridge is fairly consistent and has been divided into four distinct strata of similar characteristics within the termination depths of the borings. These strata are described briefly below.

Stratum 1 was encountered from the existing ground surface to about elevation -25 to -30 feet. The soils of this stratum predominantly consist of fine to coarse sands (USCS classification SP, AASHTO classification A-3) and silty sands (USCS classification SM) with trace gravel. Average standard penetration test (SPT N_{60}) values are approximately 30 blows per foot in Stratum 1.

Stratum 2 was encountered directly below Stratum 1 and consists of very soft to firm, slightly organic clay (USCS classification CL/CH, AASHTO classification A-6 and A-7-6) and elastic silt (USCS classification MH). Occasional thin layers of peat were also encountered. This stratum was thickest on the south side of the inlet extending to about elevation -85 to -90 feet. On the north side, the bottom of this stratum was encountered at about elevation -50 to -60 feet. SPT N_{60} values ranged from weight of hammer (WOH) to 4 blows per foot. Laboratory tests performed on representative samples of this stratum indicate that these soils are generally normally consolidated.

Stratum 3 was encountered below Stratum 2 and predominantly consists of medium-dense to very dense fine to coarse sands (USCS classification SP/SM, AASHTO classification A-1-b/A-2-4). Significant amounts of gravel were encountered within the upper portions of this stratum. Stratum 3 extended to about elevation -132 feet on the south side of the bridge to about elevation -140 to -150 feet on the north side. SPT N_{60} values in Stratum 3 ranged approximately from 10 blows per foot to more than 100 blows per foot.

Stratum 4 was encountered below Stratum 3 and predominantly consisted of loose to medium dense clayey sands and sandy clays. In our analyses, this stratum was subdivided into predominantly clayey zones and predominantly sandy zones. The upper 4 feet of this stratum consists of predominantly of sandy clay (fines contents of about 70 to 80 percent). Correlations with index property tests suggest that this zone is normally to slightly overconsolidated (OCR of 1 to about 1.25). A second, slightly stiffer, clayey zone was encountered at about elevation -145 feet on the north and south sides of the inlet. Correlations suggest that these soils are slightly overconsolidated (OCR of about 1.25 to 1.5). The soils encountered between and beneath these zones are predominantly sandy (fines contents generally less than 30 percent).

Groundwater levels were monitored over an approximate five month period. Water levels during this period generally fluctuated between about elevation +0 to +3 feet in the vicinity of the bridge.

2.0 ROADWAY RECOMMENDATIONS

The following sections provide generalized geotechnical parameters for the design and construction of the proposed SR1 roadway embankments and associated structures. The associated structures for this project include roadway embankments, mechanically stabilized earth (MSE) and modular block retaining walls, and stormwater management ponds. In addition, recommendations regarding the suitability of on-site material, compacted fill requirements, and pavements are also provided.

2.1 EARTHWORK

Placement of large embankments will be required to achieve proposed grades for the new SR1 approaches on both the north and south sides of the inlet. These embankments will extend along the centerline of the proposed roadway from approximately Station 280+00 to 293+60 on the south side of the inlet, and from Station 306+40 to 320+00 on the north side of the inlet. The proposed height of the approach embankments will be approximately 40 feet near the abutments, and will decrease with distance away from the inlet. The width of the footprint of the embankments will be approximately 112 feet (perpendicular to the proposed centerline) along their entire length. Please note that references to the centerlines of the proposed embankments in this report are referenced to the survey and construction baseline of northbound SR1 (BL Survey & Construction NB SR1).

Minimal cuts are anticipated for this project, except where the proposed embankments will overlap the west side of the existing bridge approach embankments. The soil test borings within the existing embankment indicate that these soils are predominantly sandy in nature and are likely suitable for reuse as compacted fill in other areas of the project provided that they meet the requirements outlined in Section 2.1.4 of this report and the special provisions that have been developed for this project.

2.1.1 Embankment Stability

The majority of the bridge approach embankments will be retained by MSE walls. Global stability analyses were performed for these walls and the results are presented in Section 2.3. Several smaller retaining walls ranging in height from approximately 5 to 30 feet are planned for support and grading of the planned paths and walkways on the east side of the proposed SR1 embankments. For these retaining structures and other locations where sloped fill is required to meet proposed grades, detailed stability analyses were not performed as the heights and/or locations of these walls and fills are not expected to have significant impacts on overall global stability. We anticipate that these lower height embankment fills will consist of sandy soils from nearby borrow sites that are similar in nature to the sands of Stratum 1. Our experience with similar material indicates that these soils should be stable at slopes no steeper than 2(H):1(V). Given the sandy nature of these soils, the slopes may be susceptible to erosion and some localized sloughing may occur until adequate vegetation is established. If space allows, it may be more desirable to construct slopes at 3(H):1(V) or flatter to facilitate vegetation growth and ease of maintenance.

2.1.2 Embankment Settlement

2.1.2.1 Settlement of Proposed SR1 Embankments

The large embankments of the proposed SR1 approaches are expected to induce significant consolidation settlements in the soft clays of Stratum 2. Laboratory tests were completed to estimate the compressibility and consolidation characteristics of these clays. Detailed settlement analyses along the embankment profiles were completed using the results of these tests. Assuming that regular weight fill material (125 pcf) is used in the embankments, our analyses indicate that maximum consolidation settlements of approximately 60 inches and 35 inches along the centerline of the approaches may be possible on the south and north sides of the inlet, respectively. These settlement magnitudes were estimated for the heights of fill required to achieve proposed final grade including a surcharge amount that varies as a percentage of the overall embankment height.

Our estimates represent locations near the abutments on each side of the inlet, and reflect a combination of the largest embankment heights and thickest clay sections anticipated. The

magnitudes of the estimated settlements decrease with distance away from the inlet as the embankments decrease in height and/or the thickness of the clay layer decreases. In general settlements on the order of 10 to 12 percent of the embankment height are anticipated on the south side of the inlet and about 4 to 7 percent of the embankment height on the north side. Our analyses were performed using the Boussinesq distribution to estimate stresses beneath the new embankments and conventional strain based equations to estimate the magnitude of primary consolidation settlement. Table 2.1 summarizes the results of these analyses.

Table 2.1 – Summary of Estimated Settlement Under Proposed SR1 Embankments

Location	Station	Representative Boring	Embankment Height (ft)	Surcharge Height (ft)	Estimated Consolidation Settlement (in)	
					CL SR1[1]	Edge SR1[2]
South Approach	281+00	BI-1	3	1	10	5
	283+00	BI-2	4	2	15	10
	285+00	BI-3	10	4	30	15
	287+00	BII-12	16	6	40	25
	289+00	BII-14	24	6	45	35
	291+00	BI-5	33	6	55	30
	293+00	BII-18	40	8	60	40
	293+60 (South Abutment)	BII-18	42	8	40	25
North Approach	306+40 (North Abutment)	BII-22	41	8	20	15
	307+00	BI-8	38	8	35	25
	308+00	BI-8	35	8	30	20
	310+00	BII-25	27	6	25	15
	312+00	BI-9	19	5	15	10
	314+00	BI-10	11	4	10	5
	316+00	BI-11	6	2	2	1
	318+00	CII-16	2	1	1	< 1

[1]Settlement under centerline of new SR1 embankments.
[1]Settlement under the western edge of new SR1 embankments.

2.1.2.2 Settlement of Proposed SR1 Embankments Due to Bike Path Fill

It is our understanding that upon the completion of filling operations for the proposed SR1 embankments, construction of proposed bike paths and walkways will begin directly adjacent to the east sides of the newly constructed SR1 embankments. These bike path fills will extend approximately from Station 285+00 to 293+00 and Station 307+00 to 312+00 on the south and north sides of the inlet, respectively. The filling operations associated with the construction of these bike paths and walkways will likely induce additional settlements in the Stratum 2 clays beneath the new SR1 embankments. The effects of this bike path fill will be most pronounced at the eastern edge of the new SR1 embankments, decreasing towards the center of the of the new SR1 embankments.

We understand that the bike path fill will range in height approximately from 10 to 17 feet along the south approach and 10 to 13 feet along the north approach, increasing in height towards the inlet. The geometry of this fill varies somewhat varies along the bike path alignment; however in our analysis it was assumed to be a uniform strip fill with height varying per station. Our analyses indicate that the settlements induced by the bike path fill on the south approach may potentially have an impact on the pavements on the newly constructed SR1 roadways. In addition, it may take several years for a majority this settlement to occur if no form of ground improvement is employed. On the north approach however, our analyses indicate that the bike path fill may not induce settlements large enough to have a significant impact on the new SR1 roadway pavements. Anticipated magnitudes of consolidation settlement induced by the proposed bike path fill were estimated as described previously, while the time rates of consolidation were estimated using Terzaghi's one-dimensional theory of consolidation. The results of our analyses are summarized in Table 2.2 below.

Table 2.2 – Summary of Estimated Settlement Under Proposed SR1 Due to Bike Path

Location	Station	Approximate Bike Path Fill Height (ft)	Estimated Consolidation Settlement (in)			Estimated Time for 90% Consolidation[4]
			CL SR1[1]	CL NB SR1[2]	Edge SR1[3]	
South Approach	288+00	10	1	3	7	9 months
	291+00	17	1	3	6	9 months
North Approach	308+00	13	< 1	1	2	N/A
	310+00	13	< 1	1	2	N/A

[1] CL SR1 = Centerline of new SR1 embankment
[2] CL NB SR1 = Centerline of northbound lanes of new SR1 embankment
[3] CL SR1 = Eastern edge of new SR1 embankment
[4] Time rate of settlement estimated assuming ground improvement is employed as described in Section 2.4.3 of this report

2.1.2.3 Settlement of Proposed Access Roads

The proposed SR1 approach embankments are also expected to induce significant settlements along the planned Access Road B in the southwest quadrant of the project. The current construction schedule calls for Access Road B to be constructed prior to the placement of fill for the new SR1 embankments. The plans provided by RK&K indicate that this new roadway will be approximately 30-feet wide and the eastern edge of pavement will be about 15 feet from the western face of the MSE wall supporting the south approach embankment. Fills for Access Road B are anticipated to be around 4 feet, resulting in 2 to 4 inches of anticipated settlement.

Our analyses indicate that additional settlements of up to 16-inches may be induced under Access Road B by the placement of the adjacent SR1 approach embankment at its highest point. As a general approximation, the induced settlements beneath the Access Road B can be estimated as 3 to 4 percent of the adjacent SR1 embankment height. In addition, it may take several years for a majority this settlement to occur if no form of ground improvement is employed. However, our analyses indicate that if ground improvement techniques are employed as described in Section 2.1.4 of this report, the time for 90% of the estimated consolidation settlements underneath Access Road B to occur may be reduced to approximately 8 to 9 months. The results of our analyses are presented in Table 2.3 below.

The proposed SR1 approach embankments are not expected to induce significant settlements beneath access roads A or C as the distances between the roads and embankment are relatively large and/or the thickness of the clay layer is relatively small.

Table 2.3 – Summary of Estimated Settlement Under Proposed Access Road B

Location	Station	Nearest Boring	Access Road B Fill Height (ft)	SR1 Fill & Surcharge Height (ft)	Estimated Consolidation Settlement Under Access Road B (in)
South Approach	281+00	BI-1	4	4	6
	283+00	BI-2	4	6	7
	285+00	BI-3	4	14	10
	287+00	BII-12	4	22	15
	289+00	BII-14	4	30	15
	291+00	BI-5	4	39	20
	293+00	BII-18	4	48	15

2.1.2.4 Settlement of Existing SR1 Due to Proposed SR1 Embankments

During the period while the new SR1 embankments are being constructed, the existing SR1 will be in service maintaining traffic across the inlet. The filling operations associated with the construction of the new SR1 embankments will induce some settlements in the Stratum 2 clays underneath the existing SR1 embankments. We understand that this may cause damage to the pavements on existing SR1, requiring pavement overlays at various times during construction to maintain existing grade.

To aid in the estimation of pavement overlay quantities, we have estimated the magnitudes of settlement which may be induced under existing SR1 embankments by the proposed SR1 fill. Also, it should be noted that our analyses indicate that it may take approximately 9 months for 90% of the estimated consolidations settlements under existing SR1 to occur after the placement of the proposed SR1 fill (assuming that ground improvement methods are employed as described in Section 2.1.4 of this report). Therefore, depending on the timing of the planned construction

sequence as it relates to maintenance of traffic across the inlet, these settlements should be considered an upper limit in the estimation of pavement overlay quantities. The results of our analyses are summarized in Table 2.4 below.

Table 2.4 – Summary of Estimated Settlement Under Existing SR1 Due to Proposed SR1

Location	Station	Proposed SR1 Fill Height (ft)	Surcharge Height (ft)	Estimated Consolidation Settlement (in)	
				CL SB SR1[1]	CL NB SR1[2]
South Approach	281+00	3	1	1	< 1
	293+00	4	2	1	1
	285+00	10	4	2	1
	297+00	16	6	13	5
	289+00	24	6	12	5
	291+00	33	6	13	5
	293+00	40	8	12	5
North Approach	307+00	38	8	2	1
	308+00	35	8	2	1
	310+00	27	6	2	1
	312+00	19	5	1	< 1

[1] CL SB SR1 = Centerline of existing southbound SR1 roadway
[2] CL NB SR1 = Centerline of existing northbound SR1 roadway

2.1.2.5 Time Rate of Settlements

Due to the highly plastic nature of the Stratum 2 clays, the full magnitude of the anticipated settlements may take years to occur unless some form of ground improvement is employed. Our analyses indicate that if regular weight fill material (125 pcf) is used in the embankments, and no form of ground improvement is employed, it may take over 40 years for 90 percent of the estimated settlements to occur, meaning that several inches of settlement may occur beneath the highest portions of the embankments after construction is complete. Our ground improvement recommendations, discussion of its impacts on the estimated time rate of settlement, and a summary of the results of our analyses are presented in Section 2.4 of this report.

In these analyses, the assumption was made that the consolidating layers are homogeneous, but anisotropic (meaning they behave differently in different directions). Based on the in-situ pore pressure dissipation testing performed in the consolidating layers, it was assumed that the coefficient of consolidation in the horizontal direction is at least twice as large as that in the vertical direction. In addition, the presence of thin sand layers, which can sometimes be missed by standard penetration testing, can greatly increase the time rate of consolidation. However, no such layers were indicated by the CPT soundings performed at the site.

2.1.2.6 Secondary Compression Settlements

Cohesive soils are known to exhibit creep, or secondary compression, which is a continuing settlement under sustained load following the dissipation of excess pore pressures. The mechanisms of secondary compression are not fully understood, but are believed to be at least partially associated with reorientation of clay particles over time. The rate of secondary compression is generally non-linear and decreases over time; however, this phenomenon can result in significant post-consolidation settlements of soft and/or organic cohesive soils. Our analyses based on consolidation test results indicate that secondary compression could account for as much as 1 to 2 inches of additional settlement over a 50 year period following the end of primary consolidation.

2.1.3 Suitability of Material

The majority of the on-site materials, with the exception of those found in the upper 2 to 3 feet in the wetlands areas, appear suitable for reuse as compacted fill. Based on our review of the plans for this project it appears that there will be a materials deficit and significant quantities of off-site borrow material will be required. Both on-site and off-site borrow materials should meet the requirements for compacted fill specified in Section 2.1.4 of this report and the special provisions developed for the project.

We understand that DelDOT wishes to use fly ash in the core of the SR1 embankments (i.e. between the MSE reinforcement strips). We recommend that fly ash may be used in the embankment cores, provided proper measures are taken during and after construction to prevent infiltration of both groundwater and surface water into the material. We recommend that the guidelines proposed in the Federal Highway Administration's (FHWA) *Demonstration Project*

116 – Ground Improvement Summaries, *Volume I* (FHWA-SA-98-086R) be followed in the use of fly ash in embankment construction in addition to the requirements for compacted fills specified in Section 2.1.4 of this report and the respective project special provisions.

The effects of using the lighter weight of fly ash on estimated settlements were not considered in the analyses presented herein. However, relative to typical select fill materials, fly ash is lighter and has similar if not greater strength properties (FHWA-SA-98-086R). Therefore, the use of fly ash in the cores of the proposed SR1 approach embankments may reduce anticipated settlements, and enhance global stability.

2.1.4 Compacted Fills

A significant amount of fill will be required for embankment and retaining wall construction. The quality of the fill placed for this project is an important part of its successful construction. Therefore, all earthwork operations and fills used for this project should comply with Section 202 – Excavation and Embankment, of the latest edition of the DelDOT *Standard Specifications for Road and Bridge Construction.* MSE walls typically require higher quality backfill than allowed in general filling operations. The MSE wall supplier should provide the specifications for MSE backfill material in accordance with DelDOT specifications and the special provisions developed for the project.

Review of the test boring records and laboratory test results indicate that the majority of the on-site Stratum 1 soils are suitable for use as compacted fill. In general, the clayey soils located in the upper 2 to 3 feet with in the wetlands areas are not considered suitable for use as compacted fill and should be either used in landscaping areas or be disposed of.

Before filling operations begin, representative samples of each proposed fill material should be collected. The samples should be tested to determine the standard Proctor (AASHTO T 99) maximum dry density, optimum moisture content, natural moisture content, gradation, and plasticity of the soil. These tests are needed for quality control during compaction and also to determine if the fill material is acceptable. Acceptable fill types include GM, GC, SM, SC, and ML with more than 35 percent granular material.

Prior to placing any new fill, existing topsoil, root matter, and vegetative material should be removed. Any loose, soft, or deleterious material should be undercut and replaced with compacted fill. We recommend that areas on which fill is to be placed be scarified and compacted prior to fill placement. The surface should then be proofrolled with a heavily loaded dump truck to determine if the surface is capable of supporting additional fill. Areas that rut or deflect excessively should be undercut and replaced with acceptable fill. The use of bridge lifts should only be used with the approval of the Engineer.

We recommend that compacted fill be constructed by spreading acceptable soil in loose layers not more than 8 inches thick. The soils used within embankments, around structures, in pavement areas, or for replacement in any undercut areas should be compacted to at least 95 percent of the standard Proctor maximum dry density (AASHTO T 99). The upper 24 inches of fill beneath pavements should be compacted to 100 percent of the standard Proctor maximum dry density (AASHTO T 99). The moisture content of the fill soils should be maintained within two percentage points of the optimum moisture content determined from the AASHTO T 99 Proctor density tests. Soils used in compacted fills should also be non-plastic and free of debris or fibrous organic material. Particles larger than 4 inches in diameter should not be included in the compacted fill unless approved by the Engineer.

The fill surface must be adequately maintained during construction in order to achieve an acceptable compacted fill. We recommend that the fill surface be sloped to prevent water from ponding in or on the fill. If the surface soil becomes excessively wet or frozen, fill operations should be halted, and the geotechnical engineer should be consulted for guidance.

We recommend that the fill placement and compaction be observed and documented by the geotechnical engineer. Significant deviations either from specifications or good practice should be brought to the attention of the resident DelDOT Engineer, along with appropriate recommendations. Field density tests should be performed as needed to verify that the specified degree of compaction is being achieved. The measured field densities should be modified to include material sizes used in the laboratory determination of density. Any areas that do not meet the compaction specifications should be recompacted and be retested to achieve compliance.

2.2 PAVEMENTS

It is our understanding that the pavement design for this project was performed by DelDOT's Materials Section. In order to assist DelDOT in this effort, MACTEC performed eight CBR tests (AASHTO T 180) at various locations across the site. CBR tests were performed on material at approximately planned subgrade elevation. Based on the laboratory testing program, we recommend that a design CBR value of 20 be used for flexible pavements supported on on-site material compacted in accordance with the recommendation presented in Section 2.1 of this report. This design CBR value represents approximately 2/3 of the average of all CBR test results.

Given the proposed grading for this project, off-site borrow will be required for the majority of the planned embankment construction. As such, actual pavement subgrade support values may differ from those presented above. However, we anticipate that the design parameters presented will represent the lower bound range for materials meeting the requirements specified in Section 2.1.4 and therefore should be suitable for use in pavement section design at this time. Actual CBR values should be evaluated for imported fill material, when identified, to verify the values used in design.

Preventing the infiltration of water into the subgrade is essential for the successful performance of the pavement. Both the subgrade and the pavement surface should have a minimum slope of ¼-inch per foot to promote surface drainage. Edges of the pavement should provide a means of surface water drainage by extending the aggregate base course through to side ditches or providing drain pipes. If seasonal high groundwater levels are anticipated, the use of an underdrain system may be desirable in areas where the proposed roadways are to be constructed at or near existing grade.

2.3 MECHANICALLY STABILIZED EARTH WALLS

MSE retaining walls will be used to provide support for the proposed SR1 approach embankments. These walls are intended to minimize the encroachment of the embankments into the wetlands on the west and park areas to the east side of the proposed alignment. It is our understanding that the walls supporting the south approach embankment are identified as

Retaining Wall 1 (west side) and Retaining Wall 2 (east side), while the wall supporting the west side of the north approach embankment is identified as Retaining Wall 7 (west side). The heights of these retaining walls, like the embankments which they support, are expected to be approximately 40 feet at their highest point. Plans provided to us by RK&K show that Retaining Walls 1 and 2 will be located approximately from Station 281+50 to 293+50 and Station 286+50 to 293+50, respectively. These plans also show that Retaining Wall 7 will be located approximately from Station 306+50 to 319+00.

We understand that several smaller MSE and modular block retaining walls are planned to support bike paths and walkways on the east side of the proposed SR1 embankments. As stated previously, no detailed stability analyses were performed for these walls as they are not expected to have significant impacts on overall global stability of the proposed SR1 approach embankments.

MSE walls are relatively flexible systems that can withstand significant total settlements and differential settlements as compared to other rigid wall types. MSE walls can normally withstand differential settlements of 2 inches in 100 feet. Differential settlements of 2 to 6 inches per 100 feet are often accommodated by using slip joints at regular intervals along the wall. When differential settlements exceed 6 inches per 100 feet, a two-stage construction sequence is often employed. In a two-stage construction program, the embankments are typically constructed with a temporary wire wrap facing to provide flexibility during consolidation of a compressible layer. A permanent wall facing is then installed after settlements occur thus limiting the potential for damage to the permanent facing. It has been decided by DelDOT and the Design Team that a two-stage construction program will be employed in the construction of the MSE walls supporting the proposed SR1 approach embankments.

MSE walls can be constructed on the existing Stratum 1 soils and will not require deep foundations or other forms of improvement for support. Also, the installation of wick drains and the application of surcharge to the embankments should be employed to accelerate or expedite the consolidation process. Our recommendations for ground improvement are discussed in Section 2.4 of this report.

2.3.1 Abutment Walls

It is our understanding that the proposed MSE reinforcement will wrap completely around the embankments, isolating the embankments from the abutment walls. For this reason, the abutment walls do not need to be designed to resist lateral soil pressures from the embankments. However, to protect the abutment walls in the event that some lateral soil pressure develops due to the embankments, we understand that they were designed to withstand some percentage of the coefficient of active earth pressure. Our corresponding recommendations for lateral earth pressures in the embankments are described below.

For a 40-foot high wall, we anticipate that rotation of about 1.5 to 2 inches will be necessary at the top of the wall to fully mobilize active conditions. We recommend using an active earth pressure coefficient of 0.33 assuming a sandy backfill with minimum friction angle of 30 degrees and unit weight of 125 pounds per cubic foot.

If the top of the wall is not permitted to rotate or translate, or in areas where corners are fixed, the walls should be designed to withstand at-rest earth pressures. We recommend that an at-rest earth pressure coefficient of 0.5 be used in these cases. Where rotation or translation is between 0 and 1.5 inches, then earth pressures will be between the at-rest and active state.

We recommend that passive earth pressures be neglected in design of the abutment walls due to the potential for scour and the magnitude of movement required to fully mobilize these pressures.

2.3.2 Stability

The bearing capacity of the proposed embankments was evaluated using several methods, including methods described by Bowles (1996) and FHWA's *Mechanically Stabilized Earth Walls and Reinforced Soil Slopes – Design and Construction Guidelines*, Publication No. FHWA-NHI-00-043. A minimum required factor of safety of 2.5 was used in our bearing capacity analyses.

The global stability of the proposed embankments was evaluated using PCSTABL computer software. This program is a PC-based, two-dimensional, limit-equilibrium slope stability analysis program developed at Purdue University for the Federal Highway Administration. We also

utilized GSTABL7© version 2.004 with STEDwin© version 3.59 as a pre- and post-processor for data entry and producing output graphics. A minimum factor of safety of 1.3 was used in our global stability analyses.

Our analyses were performed for locations along both the north and south approach ramp embankments using the interpreted subsurface conditions at each area. Further, the analyses were performed for wall heights ranging from 46 to 48 feet (depending on location), which includes the maximum final height of these walls and an additional fill height to account for a temporary surcharge load. Our analyses assume a horizontal backfill behind the wall. The length of the wall reinforcement was assumed to be 70 percent of the maximum wall height. Based on the width of the approach embankments (approximately 112-feet), there will be no reinforcement overlap between the east and west sides.

Our analyses assume that some strength gain will occur in the Stratum 2 clays as they consolidate under the proposed SR1 embankment fills. We understand that the construction of the proposed SR1 embankments is currently planned to occur over a minimum of approximately 230 days according to information provided to us by RK&K. In our analyses, the construction of the proposed SR1 Embankments was conservatively assumed to occur at a constant rate over a 180 period. The degree of consolidation at different times during construction was then estimated using a method for estimating time rates of consolidation under staged embankment construction proposed by Olson (1977). The degree of consolidation was then assumed to be proportional to the strength gain under full consolidation estimated using the SHANSEP method proposed by Ladd (1991). In addition, the effect of the proposed SR1 embankments on the strength of the Stratum 2 clays was assumed to decrease linearly with distance away from the embankments. This means that outside of the estimated zone of influence of the proposed embankments (approximately 100 feet according to our analyses), no increase in strength of the Stratum 2 clays was assumed in our analyses.

The results of our analyses indicate that the proposed MSE embankments meet the criteria for minimum factors of safety for global stability and bearing capacity. The MSE wall supplier should confirm stability requirements based on the actual wall configurations planned.

2.3.3 Scour

MACTEC was provided with a Scour Analysis Evaluation report prepared by Duffield Associates Inc. and dated January 21, 2005. Based on this report, we understand that maximum scour depths at the MSE walls supporting the proposed SR1 embankments are estimated at 4.4 feet and 6.2 feet for the 100-year and 500-year events respectively. These depths were estimated by assuming that the scour at the MSE walls will be equal to the contraction scour at the pier and abutment foundation.

It is our understanding that scour protection measures have been developed by RK&K in conjunction with Duffield Associates Inc. These protection measures include the installation of geotextile blankets and soil-filled geotextile bags referred to as "scour pads", and "armor tubes" respectively.

Based on our review of the details and special provisions for the scour protection provided to us by RK&K, the installation of the scour pad and armor tubes seem to be a reasonable means of protecting the MSE wall embankments from the effects of scour. As such, we have not included scour in our stability analyses.

2.3.4 Construction Considerations

Based on the results of our analyses, it appears that the approach embankments and MSE walls can be constructed in a continuous manner for the full heights of the walls; provided that the limits on filling rates and procedures for settlement monitoring specified in the plans and special provisions are observed. The settlement estimates under the edge of the proposed SR1 embankments presented in Table 2.1 indicate that 3 to 5 inches of differential settlement at the wall face is possible over a 100-foot length of wall. Due to the magnitude of settlements anticipated (and potential variability between predicted and actual settlements), it has been decided by DelDOT and the Design Team that slip joints should be incorporated into the facing panels to accommodate these settlements. DelDOT and the Design Team have also agreed that the walls will be constructed in two stages, with a temporary facing as described previously. The permanent wall facing may be installed once sufficient settlements have occurred and the potential for damage to the final facing is minimized.

2.4 GROUND IMPROVEMENT

As previously discussed, significant settlements are anticipated along the planned roadway alignment due to the placement of large embankments. The soils beneath these embankments will require some form of in-situ ground improvement in order to minimize the potential negative impacts that the anticipated magnitudes and durations of the settlements may have on the project. Several methods of ground improvement are feasible for the site; however DelDOT and the Design Team have agreed that the use of surcharging and the installation of prefabricated vertical (PV) drains will provide the most effective and economical means of ground improvement for the project. These ground improvement techniques are discussed briefly in the following sections.

2.4.1 Surcharge Loading

Surcharging is the process of loading the compressible soils in excess of the final service loads, usually by temporarily increasing the height of embankment fill. It is often used in conjunction with PV drains to increase the time rate of consolidation of the compressible layer. Surcharging has the added benefit of allowing the compressible soils to be overconsolidated (if desired) so that settlements upon reloading will generally be less than virgin compression. Surcharging also reduces long-term secondary compression since the sustained embankment loads are somewhat less than the new preconsolidation pressure of the compressible layer.

2.4.2 Prefabricated Vertical Drains

Prefabricated vertical (PV) drains, or wick drains, are thin rectangular plastic strips surrounded by a geotextile filter fabric. These strips are inserted vertically into and through compressible cohesive soil deposits to provide significantly shortened drainage path lengths that can greatly accelerate consolidation settlement time rates. PV drains are typically about 4 inches wide by ½ inch thick. The plastic core is typically wrapped in non-woven geotextile fabric, and contains channels or ridges that allow water to flow freely from the clay layer to more permeable layers above and/or below. The drains are typically installed in triangular or rectangular patterns beneath embankment fills. Drain spacing ranges from about 3 to 12 feet; however, a spacing of 4 to 6 feet is most common. The PV drains do not directly provide additional strength to the soft compressible layer; however, they do significantly shorten consolidation time, which results in an increase in strength of the layer following the dissipation of excess pore water pressures.

2.4.3 Ground Improvement Recommendations

As stated previously, DelDOT and the Design Team have agreed that the use of surcharging and the installation of PV drains will provide the most effective and economical means of ground improvement for the project. We have analyzed the benefits of using PV drains at a 6-foot triangular spacing. Surcharge loads of 8 feet at the highest points of the embankment and tapering to about 1 foot near the lowest points were considered. Regular-weight aggregate fill (125 pcf) was evaluated for use as embankment fill. In our preliminary analyses, estimated embankment settlements for lightweight aggregate fill were still substantial and thus the additional costs of using this material are likely not justified. Therefore, we did not include the use of lightweight fill in our final settlement analyses.

Settlement evaluations were completed for several locations along each of the embankments. Time rate calculations were performed using Terzaghi's one-dimensional consolidation equation in conjunction with the methods described in FHWA's *Prefabricated Vertical Drains – Vol. 1: Engineering Guidelines*, Publication No. FHWA-RD-86-168. The results of our analyses are summarized in Table 2.5.

Table 2.5 – Summary of Time Rate of Consolidation Analyses

Station	Estimated Time for 95% Consolidation (months)		
	Regular Fill Only	Regular Fill + Surcharging	Regular Fill + Surcharging + PV Drains
285+00	720	300	6
289+00	680	350	7
293+00	680	410	8
307+00	150	90	7
310+00	120	70	7
314+00	20	8	4

As shown, the use of PV drains and surcharging can greatly reduce the time necessary for the compressible clay layer to consolidate under planned embankment loads. Based on our review of the subsurface profile along the approach embankments, it appears that PV drains and surcharging should be performed beneath the entire south approach fill and from the north abutment to about Station 314+00 on the north side approach. The PV drains should also extend west beneath the Access Road B and east beneath the planned bike path fill on the south approach

embankment. Also, the PV drains should also extend west approximately 10-feet beyond Retaining Wall No. 7, and on east beneath the planned bike path fill on the south approach embankment. The time required to complete primary consolidation will vary depending on the height of the fill, surcharge load, and thickness of the underlying clay layer. Details of the PV drain and surcharging program have been developed in conjunction with RK&K and are provided in the plans and special provisions for the project.

2.4.4 Instrumentation and Monitoring

Instrumentation and monitoring of the approach embankment fills is recommended during and after construction to evaluate the magnitudes and rates of settlement. This information will be critical in evaluating the stability of the embankments during construction and determining the time at which the surcharge material may be removed. Details of the instrumentation program have been developed in conjunction with RK&K and are provided in the plans and special provisions for the project.

The proposed instrumentation program includes the installation of settlement plates, vibrating wire piezometers, and vertical inclinometers at select location in order to characterize the vertical and horizontal displacements associated with the construction of the proposed SR1 embankments and other structures. In general these instruments will be installed in groups which run perpendicular to the centerline of the proposed embankments. Three-instrument groups will be installed on either side of the inlet at various stations to provide information regarding trends in the settlement of the proposed embankments both parallel and perpendicular to their centerline.

In addition to the instrumentation proposed above, we recommend that survey points (PK nails) be established at a minimum spacing of 300-feet along the face of the MSE walls, and at the instrument bank locations described above. These points will allow evaluation of the deflections of the wall face and can be used to aid in the evaluation of data from the other instrumentation which is more likely to be damaged during construction.

Protocol for the installation, collection of data, and data evaluation is defined in the project special provisions. In general, the contractor will install the instruments, DelDOT will take the instrument readings, and DelDOT's geotechnical consultant should evaluate the data and make recommendations concerning their impact to the filling operations.

3.0 BRIDGE SUBSTRUCTURE RECOMMENDATIONS

3.1 EVALUATION OF FOUNDATION ALTERNATIVES

Based on the loading conditions imposed by the bridge on the foundations, and due to the compressible clay layer underlying the site (Stratum 2), we recommend that the proposed piers and abutments be supported by deep foundations. Several deep foundation types are feasible for support of the bridge, however it has been decided by DelDOT and the Design Team that drilled shafts, open-ended steel pipe piles, or some combination of the two types of foundation elements be used to support the bridge. These foundation elements will bear in the dense sands of Stratum 3. The following subsections present our understanding of the design considerations for the bridge substructures, summaries of our analyses, and recommendations regarding each of the proposed foundation systems for the project.

3.1.1 Axial Capacity

Each pier (arch base) foundation for the new bridge will be required to support vertical axial compressive loads of approximately 80,000 to 90,000 kips under service loading conditions. Also, temporary construction loads applied to the piers will require the foundations to support vertical axial compressive loads of approximately 110,000 to 120,000 kips. These construction loads will result in the some of the elements of each pier foundation being loaded up to 125% of their estimated allowable axial compressive capacities as recommended herein. The application of this 125% allowable stress factor to the foundations has been agreed upon by DelDOT and the Design Team as an acceptable construction loading scenario. We have evaluated the geotechnical implications of the 125% allowable stress factor, and summarized our findings in MACTEC's letter on the subject issued to FIGG on January 28, 2005.

Each abutment foundation for the new bridge will be required to support vertical axial compressive loads of approximately 30,000 to 35,000 kips under service loading conditions. During construction, temporary vertical axial compressive loads of approximately 35,000 to 37,000 kips will be applied to the abutment foundations. Based on the results of our axial capacity and settlement analyses presented in this report, we conclude that the 125% allowable stress factor as applied to the pier foundations be considered acceptable for application to the

abutment foundations as well. However, FIGG should evaluate the impact of the estimated additional settlements on the bridge superstructure before proceeding with the application of the increased temporary loading.

Additional loading conditions affecting the foundations such as group effects, scour, downdrag, and pressures due to lateral soil movements will be discussed in subsequent sections of this report.

Drilled shaft foundation capacities were evaluated using several published methods including those recommended in the Federal Highway Administration's (FHWA) *Drilled Shafts: Construction Procedures and Design Methods* (FHWA-IF-99-025) and the Transportation Research Board's (TRB) *NCHRP Report 343 – Manuals for the Design and Construction of Bridge Foundations* (1991). In addition, several "direct" methods of estimating drilled shaft capacity based on CPTu data were evaluated including methods proposed by Fiorvante et al. for end bearing capacity and Almeida, et al.; Takesue, et al.; and Elslami & Fellenius for side shear capacity as presented by Mayne (2001, & 2003).

A significant increase in reported load tests of large diameter drilled shafts has occurred over the past ten years resulting in a greatly expanded database of foundation performance. This recent data has shown that current design methods typically underestimate drilled shaft capacities in dense to very dense materials. Researchers have proposed various recent modifications to the current design procedures to attempt to account for these discrepancies, including the designation of intermediate geomaterials (materials with SPT resistance values greater than 50 blows per foot) by O'Neill and modifications for gravelly soils proposed by the Utah Department of Transportation. These modified methods result in increased design capacities; however, they still often underestimate actual drilled shaft capacities as determined by load testing.

The results of these methods represent a wide range of estimated capacities. Based on discussions with DelDOT, FHWA, and the Design Team, it was agreed that the best approach for determining drilled shaft capacities would be to utilize the upper range of values as estimated by the various design procedures with the understanding that actual capacities based on load testing at the subject site would still likely exceed these values. As such, the values used in design were developed by averaging the capacities as estimated by the Reese and O'Neill (1998) method and the Reese and Wright (1977) method.

Driven pile capacities for the sizes of steel pipe piles being considered for this project were evaluated using FHWA's *Design and Construction of Driven Pile Foundations Workshop Manual – Volume I* (FHWA-HI-97-013). The computer program DRIVEN 1.2 was used to perform these analyses. Based on discussions with FHWA, and our experience with driving piles in similar subsurface conditions, we have assumed that the steel pipe piles can be installed open-ended and then augered out to the specified depth prior to concreting. Though the pipe piles should be able to be driven open ended without significant difficulty, it was assumed in our analyses that the piles will develop a fully plugged condition with respect to ultimate end bearing resistance.

Drilled shaft and steel pipe pile allowable axial capacities and respective tip elevations are presented for each substructure in Table 3.1. Table 3.1 also presents capacities estimated for the 100-year and 500-year scour events (scour is discussed in section 3.4.1 of this report). Allowable uplift capacities for the abutment foundations are also presented in Table 3.1 (uplift is discussed in Section 3.1.2 of this report). Also presented in Table 3.1 are the estimated downdrag loads for each type of foundation element at the abutments. The recommended allowable capacities at the abutment foundations include reductions for downdrag as discussed in Section 3.1.3 of this report. A factor of safety of 2.0 was used in developing the allowable axial capacities based on the assumption that a load test program similar to those recommended in Sections 3.2.3.3 and/or 3.2.3.5 of this report is implemented. A factor of safety of 3.0 was used in developing the allowable uplift capacities as discussed in Section 3.1.2 of this report.

Table 3.1 – Summary of Foundation Recommendations

Location	Foundation Type	Tip Elevation (ft)	Est. All. Capacity (kips)[3]	Est.Cap. (100 Yr Scour) (kips)[3]	Est.Cap. (500 Yr Scour) (kips)[3]	Est. All. Uplift Cap. (kips)	Est. Downdrag Load (kips)[4]
South Abutment	6 ft D.S.[1]	-110	1280	1460	1470	800	275
	48 in P.P.[2]	-95	1850	1850	1850	460	130
South Pier	8 ft D.S.[1]	-110	3040	2800	2660	N/A	N/A
	72 in P.P.[2]	-100	3200	2870	2720	N/A	N/A
North Pier	8 ft D.S.[1]	-95	3300	3000	2830	N/A	N/A
	72 in P.P.[2]	-95	3200	3200	3200	N/A	N/A
North Abutment	6 ft D.S.[1]	-120	1940	2230	2250	1160	380
	48 in P.P.[2]	-70	1780	1780	1780	340	90

[1] Drilled shaft capacities estimated as average of O'Neill (FHWA) and Reese & Wright (NCHRP) SPT Methods
[2] Steel pipe pile capacities estimated using computer program DRIVEN 1.2
[3] Estimated allowable capacities at abutments include the effects of downdrag
[4] Estimated downdrag loads have been reduced for the effects of group interaction (and bitumen coating for steel pipe piles)

3.1.2 Uplift Loading

We understand that the abutment foundations will be used as support anchors during construction of the arch and will be subjected to vertical axial uplift loads of approximately 300 to 800 kips. As stated previously, recommended allowable uplift capacities were determined based on a minimum factor of safety of 3.0. In addition, uplift capacities for the drilled shafts were further reduced by applying a factor of 0.75 to the side resistance to account for a "Poisson effect" of the shaft under tensile loading as recommended in FHWA-IF-99-025. No additional reductions to the allowable uplift loads for steel pipe piles were made, following the recommendations in FHWA-HI-97-013.

3.1.3 Downdrag Considerations

The approach embankments for the bridge will be on the order of 40 feet high at the abutments. These fills will result in significant consolidation of the soft clays of Stratum 2. Surcharging and PV drains will be used to improve the soils and minimize post-construction settlements, however we understand that the construction schedule will require the abutments to be constructed before settlements are complete so that construction of the arch can proceed. As such, the soils of Stratum 1 and Stratum 2 will move down relative to the abutment foundation elements. This

relative movement has two effects; the foundation elements do not pick up positive compressive capacity in these strata, and the movement of the settling soils applies compressive loads to the foundation elements in addition to any structural load applied at the top of the footing. This "downdrag" load is equal to the ultimate side resistance in the settling soils.

While it is standard practice to not reduce the estimated downdrag load on a single foundation element by a factor of safety, published research indicates that downdrag loads on groups of closely-spaced elements are significantly less than downdrag loads anticipated for isolated elements. Based on published correlations and equations, our analyses indicate that due to group effects the average downdrag load on each element of the proposed drilled shaft and steel pipe pile abutment foundations may be reduced by an amount ranging approximately from 40% to 70%, depending on the dimensions of each particular group. Additional reductions in downdrag load for driven piles may be realized through the application of bitumen coatings to the surface of the piles. TRB's *NCHRP Report 393 – Design and Construction guidelines for Downdrag on Uncoated and Bitumen Coated Piles* (1997) suggests that the application of bitumen coating to the surface of driven piles may reduce downdrag loads by as much as 98%. We have assumed that bitumen coating will be applied to the surface of the steel pipe piles over their embedded length in the settling soils of Stratum 1 and Stratum 2. A reduction in downdrag load of 70% due to the effects of bitumen coating, in addition to a group effect reduction, was assumed in our analysis of the steel pipe piles for the abutment foundations.

The downdrag loads presented in Table 3.1 are the result of the reductions described above. The allowable axial compressive capacities for the abutment foundation elements presented in Table 3.1 include the effects of these reduced downdrag loads. Also, the effect of bitumen coating (i.e. a 70% reduction to the side resistance in Stratum 1 and Stratum 2) is included in the allowable uplift capacities for the abutment steel pipe piles presented in Table 3.1. However, a group effect reduction has not been applied to the allowable uplift capacities of either foundation type as the current design method of estimating group uplift capacity is conservative to begin with (FHWA-HI-97-013). Our analyses indicate that the pier foundations will not be subjected to downdrag loads due to the distance from the approach embankments.

3.1.4 Lateral Loading

The bridge foundations will be subjected to forces and deflections associated with lateral movements of the bridge superstructure and lateral movements of the subsurface soils at the pier foundations and abutment foundations respectively. Each of these conditions is described in the following sections.

3.1.4.1 Loading Due to Lateral Superstructure Movements

We understand that lateral deflections on the order of 5 inches are anticipated at the bridge piers due to shrinkage and temperature effects on the bridge superstructure. The medium-dense to dense sands of Stratum 1 will provide significant resistance to the deflections imposed at each pier. In order to reduce the force that these deflections will induce on the bridge superstructure, DelDOT and the Design Team have decided that bearings will be placed between the arch rib and the arch base footing on the south side of the inlet. These bearing are intended to accommodate the movement of the bridge superstructure in the longitudinal direction (along the length of the bridge) to significantly reduce the lateral force applied to the foundations.

3.1.4.2 Loading Due to Lateral Soil Movements

As stated previously, the approach embankments for the bridge will be on the order of 40 feet high at the abutments. According to FHWA-HI-97-013, lateral movements of soft compressible soils induced by large embankment fills may apply significant lateral pressures to nearby piles if they are installed prior to completion of the anticipated embankment settlements. The construction schedule will require the abutments to be constructed before settlements are complete so that construction of the arch can proceed. Therefore the abutment foundation elements must be designed to withstand lateral pressures resulting from the soil deflections.

We evaluated several published methods for estimating the lateral pressures applied to deep foundation elements due to lateral movements of soft soils under embankment loading. Each of the methods evaluated provides significantly different results depending on the assumptions made in the analysis. Based on our assumptions and evaluations of each method of analysis, we selected an approach that is reasonable but not overly conservative. Our approach for estimating the lateral pressures applied to the abutment foundation elements of the subject bridge includes

the mobilized shear strength method proposed by Stewart, et al. (1994) in the Stratum 2 clays and the Boussinesq distribution for stresses in an elastic half-space induced by a flexible loaded area in conjunction with Mayne and Kulhawy's equation for the at-rest coefficient of lateral earth pressure in the Stratum 1 sands. The results of our analyses are presented in Table 3.2. It is our understanding that the values presented in Table 3.2 were used by FIGG in the design of the abutment foundations.

Table 3.2 – Summary of Lateral Soil Pressures at Abutment Foundations

Location	Foundation Type	Tip Elevation (ft)	Estimated Lateral Pressure (ksf)	
			Stratum 1 Sands	Stratum 2 Clays
South Abutment	6 ft D.S.	-110	1.5	1.9
	48 in P.P.	-95	1.5	1.3
North Abutment	6 ft D.S.	-120	1.5	1.9
	48 in P.P.	-70	1.5	1.3

3.1.5 Scour

MACTEC was provided with a Scour Analysis Evaluation report prepared by Duffield Associates Inc. and dated January 21, 2005. Based on this report, we understand that maximum scour depths for the 100-year event are anticipated to be around 33.8 feet at the abutments and 27.6 feet at the piers. Scour depths for the 500-year event are anticipated to be around 39.3 feet at the abutments and 34.5 feet at the piers. These depths represent a combination of local and contraction scour. The scour analysis reports contraction scour depths of 4.4-feet and 6.2-feet for the 100-year and 500-year events respectively. The remainder of the anticipated scour depth at each foundation for the 100-year and 500-year events is caused by local scour.

It is our understanding that the bridge foundations will need to be designed based on the 100-year scour depths and checked for safety (i.e., factor of safety $\geq$ 1.0) with the 500-year scour depths. For this reason we have developed axial capacity estimates for each foundation type including the effect of scour for both the 100-year and 500-year event. These estimated capacities are presented in Table 3.1. It should be noted that the estimated axial capacities presented in Table 3.1 for drilled shafts at the abutment foundations under 100- and 500-year scour conditions are greater than the estimated allowable capacities of those same elements assuming no scour. This

is due to the fact that, under scour conditions, a portion of the soil (Strata 1 and 2) applying downdrag load to the foundation elements is removed. This trend in estimated capacities including scour is not realized for the abutment steel pipe piles, as the reduced overburden pressure used in the capacity analysis essentially negates any benefits of reduced downdrag loads.

3.1.6 Settlement

Foundation group settlements were estimated for each foundation type at both the piers and abutments using the equivalent raft method (FHWA-IF-99-025). These settlement estimates were developed for both service loading conditions (i.e. 10,000 days after construction), and construction loading conditions (i.e. load cases for which the 125% allowable stress factors are required). Settlement estimates for the sandy soils of Strata 3 and 4 were calculated using estimated using elastic modulus values based on various published correlations. Settlement estimates for the sandy clay layers of Stratum 4 were calculated using consolidation parameters evaluated from laboratory testing and published correlations. In addition, these results were compared to Meyerhof's SPT method (FHWA-HI-97-013) which estimates settlements in sands based on SPT resistance values.

The results of these analyses indicate that the majority of the estimated settlements occur due to compression of the Strata 3 and 4 sands. These settlements will be elastic in nature and will occur during or shortly after loading. Consolidation settlements within the sandy clay layers of Stratum 4 were generally estimated to be less than 1 inch and are anticipated to occur within a few months after loading. The results of these group settlement analyses are presented in Table 3.3.

Published studies have suggested that the method of analysis utilized for estimating deep foundation group settlement is less critical than the selected soil parameters (i.e., elastic modulus, coefficient of consolidation, etc.). In order to evaluate this, MACTEC performed a parametric study by varying the elastic modulus values for the Strata 3 and 4 sands within the range of published correlation values. The results of this study showed that settlements in these sands may vary approximately from 1.5 inches to 7 inches at the piers, and less than 1 inch to 4 inches at the abutments. The parameters selected for the settlement estimates presented in Table 3.3 are within the range of reasonable values based on MACTEC's experience with similar soils. Also, it should be noted that the elastic components of the estimated settlements will occur during

construction as load is gradually applied to the foundations. Therefore the settlements presented in Table 3.3 should be considered an upper range when evaluating potential differential settlement values.

Table 3.3 – Summary of Estimated Foundation Settlements

Location	Foundation Type	Settlement Breakdown	Estimated Foundation Settlement (in)			
			Max. Constr. Load Case		Day 10,000 Load Case	
			FHWA[1]	Meyerhof[2]	FHWA[1]	Meyerhof[2]
South Abutment	6-ft D.S.	Sand/Clay	2.1/0.6	0.9/0.5	1.1/0.1	0.7/0.4
		Total	2.7	1.4	1.2	1.1
	48-in P.P.	Sand/Clay	2.1/0.5	0.8/0.4	1.9/0.3	0.7/0.3
		Total	2.6	1.2	2.2	1.0
South Pier	8-ft D.S.	Sand/Clay	3.0/0.9	1.4/1.0	2.1/0.5	1.0/0.6
		Total	3.9	2.4	2.6	1.6
	72-in P.P.	Sand/Clay	3.5/1.0	1.5/1.0	2.4/0.6	1.0/0.6
		Total	4.5	2.5	3.0	1.6
North Pier	8-ft D.S.	Sand/Clay	2.7/0.7	1.2/0.5	2.2/0.5	1.0/0.3
		Total	3.4	1.7	2.7	1.3
	72-in P.P.	Sand/Clay	3.0/0.7	1.2/0.6	2.2/0.5	0.9/0.3
		Total	3.7	1.8	2.7	1.2
North Abutment	6-ft D.S.	Sand/Clay	1.6/0.2	0.6/0.2	1.7/0.2	0.6/0.2
		Total	1.8	0.8	1.6	0.8
	48-in P.P.	Sand/Clay	2.2/0.1	0.7/0.2	1.9/0.1	0.6/0.2
		Total	2.3	0.9	2.0	0.8

[1] Equivalent Raft Method - FHWA-IF-99-025
[2] Meyerhof SPT Method - FHWA-HI-97-013

3.1.7 Group Effects

The spacing and layout of the foundation systems for the bridge will have an impact on the axial and lateral capacities of the foundation elements as well as the overall vertical settlements of the structure. Group effects tend to decrease lateral resistances (increasing flexibility) as compared to individual foundation elements due to "shadowing" effects of the trailing elements.

Group effects may also have an impact on axial capacities. Generally, groups of driven piles bearing in sandy soils will have a higher capacity than the sum of the individual pile capacities (i.e., efficiency greater than 1) due to the densification of the soils in and around the pile group that occurs during driving. Drilled shaft installation on the other hand results in a temporary

decrease in confining stresses as the shafts are constructed, and group efficiencies less than 1 are possible. However, little research is available to adequately assess these effects. The limited data available suggest that some reduction of axial capacity may occur for center to center spacing of less than about five shaft diameters. The data also suggest that the reduction is just slightly less than 1 for shafts where the cap is in contact with the ground surface. Given this and the fact that current design procedures typically underestimate shaft capacities, no reductions for group effects were considered in determining allowable axial shaft capacities presented in Table 3.1.

Group effects will also affect the settlements of the foundation groups at each substructure location. Foundation group settlements can be evaluated by an "equivalent raft" or "equivalent footing" method of analysis as described in FHWA-IF-99-025 and FHWA-HI-97-013. These analyses assume that the deep foundation elements are replaced by a raft or footing foundation bearing at a depth equal to 2/3 of the embedment length into the bearing layer (or at the top of the bearing layer for end bearing foundation elements). The equivalent footing is assumed to be the same dimensions as the outer perimeter of the pile group and is assumed to support the total dead load plus sustained live loads. Foundation layouts were developed by FIGG for the foundation alternatives being considered based on the recommended allowable capacities and tip elevations presented in Table 3.1.

3.1.8 Constructability Considerations

The foundation systems planned for the piers will require experienced contractors to successfully construct. Subsurface conditions such as shallow groundwater, local artesian conditions and loose granular material may present difficulties for drilled shaft construction. Also, the requirement of applying bitumen coating to the steel pipe piles at the abutments adds complexity to the handling and installation of the piles. Therefore detailed special provisions have been developed for each foundation type that address the construction and functional capability requirements of the project.

3.2 FOUNDATION RECOMMENDATIONS

The information presented above has been considered in the evaluation of the various foundation types and in preparation of our recommendations. The following sections provide detailed recommendations for the pier and abutment foundations.

3.2.1 Pier Foundations

We recommend that the piers be supported on constant diameter drilled shafts or steel pipe piles bearing in the dense sands of Stratum 3. Based on feasibility and cost evaluations performed by the Design Team, it has been agreed by DelDOT and the Design Team that the drilled shafts be 96 inches in diameter, and that steel pipes piles be 72 inches in diameter with a 1-inch wall thickness. We recommend that these elements be installed to their respective tip elevations and allowable capacities presented in Table 3.1. Our analyses indicate that shafts and/or piles bearing at these elevations will be above the zone of influence of Stratum 4, where lenses of softer and/or looser sandy clays and clayey sands were encountered. However, we recommend that shafts or piles bear at or above elevation -120 feet to minimize the potential to induce additional settlements in the Stratum 4 soils.

3.2.2 Abutment Foundations

We recommend that the abutments be supported on constant diameter drilled shafts or steel pipe piles bearing in the dense sands of Stratum 3. Based on cost analysis performed by FIGG, it has been agreed by DelDOT and the Design Team that the drilled shafts be 72 inches in diameter, and that steel pipes piles be 48 inches in diameter with a 1-inch wall thickness. We recommend that these elements be installed to their respective tip elevations and allowable capacities presented in Table 3.1. Our analyses indicate that shafts and/or piles bearing at these elevations will be above the zone of influence of Stratum 4, where lenses of softer and/or looser sandy clays and clayey sands were encountered. However, we recommend that shafts or piles bear at or above elevation -120 to minimize the potential to induce additional settlements in the Stratum 4 soils.

The allowable capacities for the abutment foundation elements presented in Table 3.1 account for anticipated downdrag forces on the shafts. In other words, the recommended allowable capacities for the abutments represent the allowable amount of capacity that is available to resist structural loads at the top of the foundation. For more detailed discussion of the application of downdrag to the abutment foundation elements, see section 3.1.3 of this report.

The abutment foundation elements will also need to provide uplift resistance as they will be used to anchor temporary towers during placement of the precast bridge segments. Estimated allowable uplift capacities for the abutment foundation elements are presented in Table 3.1. The allowable uplift values include the buoyant weight of the shafts, but do not include downdrag forces since these will act to resist the uplift loads. For more detailed discussion of the development of our uplift capacities, see section 3.1.2 of this report.

3.2.3 Foundation Contingency Plans

In the event that load test data indicates that the recommended allowable capacities are not achieved at the specified tip elevations for the foundation elements, measures can be taken to potentially increase their capacity. The steel pipe piles may be driven deeper to increase their capacity, up to the maximum tip elevation of -120 feet recommended above. However, due to the tips of the drilled shafts already being lower than the steel pipe piles, and the decreasing density of the Stratum 3 soils as they approach Stratum 4, our analyses indicate that it may not be possible to increase the capacity of the drilled shafts by increasing their length; particularly at the south pier and abutment.

An alternative to increasing the length of drilled shafts is post grouting of the shaft tips. Post grouting of drilled shafts is a procedure in which grout tubes and a grout distribution assembly are attached to the reinforcement cage prior to placement in the shaft excavation. Once the shaft concrete has reached a specified strength, neat cement grout is pumped through the grout tubes into the grout distribution assembly at the base of the shaft until the design pressure is achieved. The soils near the base of the shaft experience densification and cementation with the introduction of the grout, thus increasing their bearing capacity.

Post grouting of drilled shafts targets the mechanisms which make drilled shaft capacity difficult to predict, especially in soils with layers or lenses of variable density like the Stratum 3 sands at the Indian River inlet site. These mechanisms include soil relaxation at the shaft bottom due to excavation, improper cleaning of debris from the shaft bottom, and strain incompatibility between side resistance and end bearing.

The introduction of pressurized grout to the shaft bottom has been shown to have significant impacts on the axial capacity which may be expected from drilled shaft foundations. Several case studies have been published which indicate increases in ultimate axial capacity ranging from 50% to 100%. Several of these case studies originated from projects in Florida, as the post grouting technology has been in use there for several years.

In addition, post grouting is a cost effective measure for increasing the capacity of drilled shafts. Contractors to whom we have spoken have indicated that post grouting of drilled shafts, similar in size to those for the abutments at the Indian River Inlet Bridge, has recently been performed for only a small percentage of the anticipated cost of each foundation for this project.

It should be noted that the post grouting procedure described herein is a patented process; however there are specialty subcontractors who are licensed to perform the procedure.

3.2.4 Foundation Construction Recommendations

The following sections present our general recommendations concerning the construction of each foundation type recommended for the Indian River project. In addition to these recommendations, special provisions for each of the proposed foundation types have been developed which describe in detail the requirements for the construction, installation, inspection, and testing of the foundations for the project.

3.2.4.1 Drilled Shaft Installation

Drilled shaft capacities are greatly affected by construction methodology. Most design procedures, including the FHWA methods, are based on empirical data from load tests of shafts installed with conventional construction methods. Inexperienced contractors and/or poor

construction techniques can result in significantly reduced capacities from those calculated by current design procedures. As discussed above, detailed special provisions have been developed for the project, specifying prequalification and/or minimal experience for drilled shaft contractors, and detailing the anticipated requirements for the successful construction of drilled shafts at the Indian River inlet.

We recommend that drilled shafts be constructed using the polymer slurry method, in combination with the use of temporary casing due to the high ground water and sandy soils at the site. The use of polymer slurry and casing is intended to stabilize the shaft excavation prior to concrete placement. Concrete will need to be placed by tremie method to reduce the potential for segregation of the aggregate when using the wet method of construction.

Localized artesian conditions were encountered on the north side of the bridge at boring BII-21 directly beneath the Stratum 2 clay (at about elevation -55 feet). In addition, gravels and cobbles were encountered in this area from about elevation -90 to -100 feet. The drilled shaft contractor should be aware of these issues and be prepared to address them if encountered during construction.

3.2.4.2 Drilled Shaft Inspection

Direct inspection of the shaft excavation is not possible when using slurry; however it is possible to use various remote inspection methods to evaluate construction quality. We recommend that shaft inspection be performed using a system such as the Shaft Inspection Device (SID) or Mini-SID developed by the Florida Department of Transportation. As with drilled shaft contractors, experienced drilled shaft inspectors are critical to the success of most drilled shaft projects. Procedures for inspecting drilled shaft construction will vary by project depending on construction methodology, the experience of the contractor's crew, subsurface conditions, load test results, etc. Therefore, it is important that the drilled shaft inspector be well experienced in various construction methodologies so that the best inspection procedures for this project can be developed to ensure the successful installation of the drilled shafts.

Close observation and monitoring by a geotechnical engineer familiar with the subsurface conditions at the site and having considerable experience with foundation installation procedures is considered necessary during construction in order to confirm that the foundations are installed satisfactorily to meet the design intent and criteria.

During shaft installation, the engineer should:

- Confirm that the shafts are within the tolerances for location and verticality
- Measure and record the shaft dimensions, including elevations of top and bottom of casing
- Record the shaft tip elevation
- Confirm that the shaft slurry or drilling fluid has been properly cleaned of cuttings prior to concreting
- Confirm that soil and ground-water intrusion is properly controlled
- Monitor concreting of the shaft for any unexpected loss of concrete into voids. The drilled shaft inspector should also log and record the soil strata encountered by the drilling process to confirm that the shafts have penetrated the anticipated load bearing materials. Reinforcing steel placed in the shafts should be checked by the geotechnical engineer prior to installation into the shaft for compliance with plans and specifications.

Particular attention is required during concrete placement in order to confirm that a structurally sound and continuous shaft has been constructed. A representative number of concrete test cylinders should be formed each day of installation, with a minimum of one set of six test cylinders per shaft for subsequent laboratory compressive strength testing.

Crosshole sonic logging (CSL) testing should be performed on all production and technique shafts. CSL test results should be reviewed by a qualified engineer experienced in the interpretation of non-destructive testing results.

3.2.4.3 Technique Shafts and Load Testing

We recommend that the contractor be required to construct one technique shaft on each side of the bridge to verify that his proposed installation method can achieve the required results. Of the technique shafts constructed, at least one should have a diameter of 72 inches, and at least one should have a diameter of 96 inches. Technique shafts should be constructed prior to installation

of dedicated load test shafts or production shafts and should utilize the same means and methods to be used for production shafts. CSL testing should be performed on these shafts prior to approving the contractor's method of installation.

Drilled shaft capacities and corresponding lengths may vary significantly based on designs from load test data versus conventional design procedures. Axial load tests should be performed to verify that the contractor's construction methods can produce the required minimum shaft capacities and confirm the design assumptions. A full-scale, instrumented load testing program for the technique drilled shafts is recommended for this project for the following reasons:

- To confirm the minimum factor of safety with regard to the recommended drilled shaft design.
- To confirm that the installation techniques employed by the selected drilled shaft contractor can successfully produce a drilled shaft that will support the anticipated design loads in accordance with the project specifications.
- To provide settlement information of the tested shafts under various test loads which may be used to confirm or refine the predicted settlement estimates for production shafts under the design working loads.

The test shafts should be instrumented with strain gauges to allow delineation of the side resistance components of the various strata. The test loads should be provided by Osterberg load cells (O-CELLs). Installation and testing should be performed under the direction of the geotechnical engineer by a specialty subcontractor experienced in O-CELL load testing. The load test results should be reviewed by the geotechnical engineer prior to approving the contractor to begin installation of production shafts.

Statnamic testing is generally not recommended by FHWA. We do not recommend that Statnamic testing be used for this project. Statnamic testing may result in significant load shed in the Stratum 1 sands (of which the capacity contribution is being neglected in design due to downdrag) for the abutment test shaft, and adequate load may not reach the base of the shaft.

3.2.4.4 Steel Pipe Pile Installation

Proper pile installation is critical to the performance of any driven pile foundation system. The cost effectiveness and safety of driven pile foundations depend on the piles being installed so as to achieve the performance requirements assumed in design. Since it is not possible to visually inspect the finished product, strict quality control must be exercised in the installation and testing of each pile (FHWA-HI-97-014).

In general, we recommend that steel pipe piles be fabricated, handled, stored, and installed as shown on the plans and described in the special provisions. We do not anticipate that the contractor will encounter significant difficulty driving piles of the recommended dimensions at the Indian River inlet, however, measures such as pre-drilling or center relief drilling may be used if approved by the engineer.

3.2.4.5 Steel Pipe Pile Inspection

The installation of large diameter piles such as those recommended for this project requires knowledgeable supervision and inspection. FHWA suggests that this is in part due to less redundancy in the foundation system as fewer of the high capacity piles are required. Therefore it is important that the inspector be knowledgeable and experienced in driving piles of the dimensions recommended herein in subsurface conditions similar to those at the Indian River inlet. We recommend that steel pipe pile inspection generally include the following items (FHWA-HI-97-014).

- Review of all design reports, project plans, and specifications.
- Inspection of piles prior to installation.
- Inspection of pile driving equipment prior to and during driving.
- Inspection during driving of production and test piles including maintenance of detailed driving records.

3.2.4.6 Steel Pipe Pile Testing

We recommend that a full-scale, instrumented load testing program be implemented for the steel pipe piles at the Indian River inlet. This load testing program should include dynamic testing of approximately 10 percent of production piles during driving and restrike, and static load testing of

one 48-inch diameter production pile at each abutment foundation location (with bitumen coating and driving collars), and one 48-inch diameter sacrificial pile at each pier foundation location (uncoated, without driving collars).

The results of the static and dynamic testing should be evaluated by the engineer to develop driving criteria and to estimate available unit side resistance and end bearing values for the 48-inch diameter test piles. The results of these tests will then be evaluated to develop driving criteria for the bitumen coated abutment piles with driving collars, and 72-inch diameter piles at the pier foundations.

4.0 OTHER GEOTECHNICAL CONSIDERATIONS

4.1 SOIL CORROSIVITY/REACTIVITY CONSIDERATION

Corrosivity and reactivity of the site soils were evaluated from laboratory resistivity, pH, chlorides, and sulfates content tests. The results of these tests indicate that the soils at the site are generally relatively neutral (pH from about 6 to 8), although resistivity values are rather low (approximately 200 to 500 ohm cm for in-situ moisture contents). Chloride contents range from about 30 to 100 ppm, and sulfates range from about 30 to 2,000 ppm. It is our understanding that the potential for corrosion of metal structures, such as MSE wall reinforcement strips and connections, and steel pipe piles or casing has been considered in the final design of the bridge foundations.

4.2 SEISMIC CONSIDERATIONS

Based on the map of horizontal acceleration (A) presented in AASHTO's Standard Specifications for Highway Bridges, 17th Edition, the project site is located in a zone with a mapped acceleration coefficient of 0.05 g. This value represents the peak horizontal ground acceleration in rock with a 10 percent probability of exceedence in 50 years (10%/50), corresponding to a recurrence interval of about 500 years. Based on the acceleration coefficients listed above the site is considered to be in a Seismic Performance Category A area. As such, no special seismic design measures are required.

4.3 IMPACTS ON ADJACENT FACILITIES

The proposed approach embankments will result in significant settlements of the Stratum 2 clays in areas not previously subjected to similar loads. There are no significant existing structures within close proximity to the planned approach embankments. Based on the locations of the proposed bridge abutments and piers, we do not anticipate significant impacts on the existing bridge or other nearby structures during construction of the bridge foundations. However, we recommend that a pre-existing condition survey be performed for the bridge and park facilities in close proximity to the proposed bridge. This survey should include photographs and

documentation of existing cracks, distress, and/or other damage to these facilities prior to initiation of construction activities. This survey can then serve as a baseline in assessing possible damage claims during construction.

4.4 TEMPORARY DEWATERING

Ground water was encountered within the upper 5 feet at most boring and temporary piezometer locations, corresponding to elevations of about 0 to +3 feet. Although most of the site will be constructed at grade or in fill, cuts for utilities, MSE leveling pads, footings, or undercuts will likely encounter ground water. The sandy soils of Stratum 1 are highly permeable and subject to collapse when saturated. As such, we anticipate that sump pumps may not be an effective means of dewatering and a system such as well points may be required.

It is our understanding that the plans call for the contractor to install temporary sheet piling through the Stratum 1 sands some depth into the Stratum 2 clays, essentially constructing a temporary cofferdam for the construction of each footing. We understand that the contractor will then dewater and excavate the space inside the cofferdams and pour a tremie seal in each to prevent seepage of the groundwater into the excavations. Intermittent pumping will then be used as a secondary means of keeping the excavations dry. Based on our experience with similar subsurface and groundwater conditions, this approach seems to be reasonable.

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