I-5 / SR-502 INTERCHANGE
FINAL DESIGN
CLARK COUNTY, WASHINGTON

Task 7.1
APPENDIX TO GEOTECHNICAL REPORT
FOR RETAINING WALLS PIER 1 LEFT AND RIGHT
AND PIER 5 LEFT AND RIGHT

Agreement Y9854
Task AA Amendment #2

Prepared for
Washington State Department of Transportation

October 20, 2006
Parsons Brinckerhoff Quade & Douglas, Inc

INTRODUCTION

The Washington State Department of Transportation (WSDOT) has retained Parsons Brinckerhoff Quade & Douglas, Inc. (PBQD) to provide final design services for the planned I-5 / SR-502 Interchange in Clark County, Washington. The scope of this work is defined in the PBQD letter to WSDOT dated June 2, 2006. Verbal notification-to-proceed was received on June 5, 2006.

Reference information prepared by WSDOT and provided to PBQD for this work includes the project Geotechnical Report[1] and concept-level plan/profile alignments[2]. The WSDOT Geotechnical Design Manual (WSDOT GDM),[3] AASHTO LRFD Bridge Design Specifications (AASHTO LRFD)[4], and references in the WSDOT GDM including the Federal Highway Administration (FHWA) Design Guidelines[5] were used as the design basis.

DESCRIPTION

This document is an appendix to the 2005 Geotechnical Report prepared by WSDOT, and contains geotechnical recommendations for Retaining Walls Pier 1 Left and Right and Pier 5 Left and Right (4 walls total). All are fill walls planned to retain roadway embankment of the approaches for the "SR 502 over I-5 Bridge".

The subject bridge is a 4-span reinforced concrete structure supported on 5 bents. The total length and width of the bridge are 720 feet and 52 feet, respectively. Each interior bent will be supported by two 10 foot diameter drilled shaft. Bents 1 and 5 form the abutments and are supported by 2 foot diameter steel piles. Embankment side slopes will be inclined at a maximum 2:1 (H:V) slope beyond the retaining walls. Bent foundations have been designed by WSDOT.

Walls Pier 1 Left and Pier 1 Right are each 60 feet in length, up to 26 feet high, and are located along the west approach between Stations M 13+00 to 14+00.

Walls Pier 5 Left and Pier 5 Right are up to 78 feet long, up to 39 feet high, and are located along the east approach between Stations M 21+00 to 22+00.

A two-stage wall system is recommended for the retaining walls at Piers 1 and 5. Currently, none of the pre-approved proprietary design is approved as a two-stage wall. However, it is anticipated that the WSDOT standard plan wall design will be modified as a two-stage wall.

SOIL CONDITIONS

Soil conditions revealed from Borings BR-01-05 and BR-05-05 were used to characterize the subsurface conditions at Retaining Walls Pier 1 Left and Right and Retaining Walls Pier 5 Left and Right, respectively. BR-01-05 was drilled to 78.3 feet below surface and BR-05-05 was drilled to 74.5 feet below surface.

In general, the soil profile underlies the Retaining Walls Pier 1 Left and Right consists of fine grained catastrophic flood deposits – sandy silts overlying silty fine sands. Boring BR-01-05 penetrated approximately 9 feet of medium stiff sandy silt and approximately 14 feet of very loose silty sand mixed with soft sandy silt. Medium stiff sandy silt to silt was encountered between 23 feet to 39 feet. Dense to very dense silty sand to poorly graded sand was penetrated between 39 feet to 64 feet over very dense well-graded gravel with sand from 64 feet to the bottom of the boring. The blow counts (N values) ranged from 2 to 9 blows per foot in the upper 39 feet and grades to 22 to 24 blows per foot between 39 feet to 59 feet. The blow count in the gravel layer is generally above 50 blows per 6 inches.

Similar soil conditions were encountered in Boring BR-05-05. Medium dense sandy silt was encountered at the ground surface to approximately 10 feet. Very loose to medium dense silty sand and medium stiff sandy silt were encountered from 10 feet to 24 feet, followed by a 5 foot layer of very soft lean clay. Loose silt and medium dense silty sand and poorly graded sand underlie the clay from 29 feet to 59 feet. Very dense well graded gravel with sand was encountered from 59 feet to bottom of the boring. The blow counts (N values) ranged from 4 to 13 blows per foot in the upper 24 feet and grades to 2 blows per foot in the very soft lean clay. The blow counts in the silty sand layer ranged from 20 to 28 blows per foot. The blow count in the gravel layer is generally above 50 blows per 6 inches.

GROUNDWATER CONDITIONS

A 1-inch diameter open standpipe piezometer was installed in Boring BR-01-05. Following installation, the depth to ground water in the piezometers was measured by WSDOT in September of 2005 and October of 2005. Data collected revealed consistent groundwater elevation at 19.2 feet below surface. However, the groundwater level could change due to seasonal weather fluctuations.

SEISMIC DESIGN

The site will behave as Site Class E as described in Table 6-3 of the WSDOT GDM (average blow count less than 15). With a peak bedrock acceleration of 0.2g per the 2005 Geotechnical Report, the resulting peak horizontal ground surface acceleration is 0.25g.

RECOMMENDED DESIGN PARAMETERS

The soil parameters listed in Table 1 should be used to design the retaining walls in accordance with Sections 15.4.2.7 and 15.4.2.9 of the WSDOT GDM. As indicated in Section 15.4.2.9 of the WSDOT GDM, walls and abutment that are free to translate or move during a seismic event may use a reduced horizontal acceleration (k_h) of approximately ½ of the peak horizontal ground surface acceleration. Vertical acceleration (k_v) should be set equal to 0.

Table 1 RECOMMENDED DESIGN PARAMETERS

Parameter	Value
Backfill Soil Unit Weight, γ (pcf)	120
Backfill Soil Buoyant Unit Weight, γ' (pcf)	60
Backfill Soil Friction Angle, ϕ (degrees)	32
Active Lateral Earth Pressure Coefficient , K_a (–)	0.3
Passive Lateral Earth Pressure Coefficient , K_p (–)	3.0
Peak Horizontal Ground Surface Acceleration, PHA (g)	0.25
Seismic Active Earth Pressure Coefficient , K_{ae} (–)	0.6
Reinforced Backfill Soil Unit Weight, γ (pcf)	130
Reinforced Backfill Soil Friction Angle, ϕ (degrees)	34
Foundation Soil Unit Weight, γ_f (pcf)	120
Foundation Soil Buoyant Unit Weight, γ' (pcf)	60
Foundation Soil Friction Angle, ϕ (degrees)	32

EARTH PRESSURE

The recommended soil parameters listed in Table 1 should be used to determine earth pressures acting on the retaining walls in accordance with WSDOT GDM Section 15.4.2.7. The active lateral earth pressure coefficient (K_a) listed in Table 1 is based on Coulomb theory. It should be used in the retained soil and applied as a triangular load to the base of the wall. The static active earth pressure resultant (P_a) should be applied at ⅓ H from the bottom of the pressure distribution, where H is the height of the wall.

Passive lateral earth pressure coefficient (K_p) contributed by the upper 2 feet of the level backfill at the toe of the wall should be neglected unless the soil is confined or protected by a concrete slab or well drained pavement. Similarly, the passive lateral earth pressure coefficient contributed by the upper 3 feet of soil should be neglected for a descending slope at the wall toe, regardless of any surface protection.

The seismic active earth pressure coefficient (K_{ae}) in Table 1 was determined by the Mononobe-Okabe method. AASHTO LRFD requires the combined static and seismic lateral earth pressure force to be applied at mid-height of the wall, from the bottom of the pressure distribution, resulting in a uniform pressure distribution in which the pressure is equal to $0.5K_{ae}\gamma H$.

SURCHARGE LOADS

The wall should be designed for surcharge loads due to surface loads within the backfill. At a minimum, a uniform vertical pressure of 250 psf should be considered in the design. This shall be multiplied by the appropriate earth pressure coefficient to determine the resulting horizontal pressure on the wall. The lateral load will act uniformly along the face of the wall. Other loads can be evaluated using the procedures of AASHTO LRFD Section 3.11.6 on a case-by-case basis.

GEOTECHNICAL ANALYSIS

Geotechnical analyses performed on the MSE walls with respect to failure modes included an evaluation of bearing capacity, overall stability, overturning stability, sliding stability, and settlement estimate. The following design criteria referenced from MSE Walls and Reinforced Soil Slopes, FHWA Publication No. NHI-00-043, was used in the design.

Bearing Capacity	F.S. ≥ 3.0
Overall Stability	F.S. ≥ 1.5
Overturning Stability	F.S. ≥ 2.0
Sliding Stability	F.S. ≥ 1.5
Seismic Stability	F.S. ≥ 75% of static F.S. (for all failure modes)

Bearing Capacity

Meyerhof's bearing capacity equation was used to calculate the bearing capacity of the foundation soil under the retaining walls at Piers 1 and 5. Ultimate or "nominal" bearing capacity (q_n) of the cohesionless foundation was computed as

$$q_n = 0.5\gamma_f N_\gamma B'$$

where B' is the effective footing dimension (i.e. effective footing width (B-2e))

e is the eccentricity

$$e = \frac{lengthofre\inf orcement}{2} - \frac{(\sum M_R - \sum M_o)}{weightofretainingmass}$$

Based on the foundation soil conditions, a an ultimate or "nominal" bearing capacity (q_{ult} or q_n) of 8.5 tons per square foot at Pier 1 and 17 tons per square foot at Pier 5 may be used to assess the walls at Pier 1 and Pier 5, respectively. Modifications to q_n for high groundwater level are provided in section 4.4.7.1.1.6 of AASHTO LRFD.

Overall Stability

The overall stability of the retaining walls was assessed using limit equilibrium methods in accordance with WSDOT GDM Chapter 7. The reinforced soil wall was modeled as a rigid body and only failure surfaces completely outside the reinforced mass were considered.

Based on the anticipated subsurface conditions and the proposed MSE wall heights of 26 feet at Pier 1 and 39 feet at Pier 5, the overall stability of the proposed MSE walls exceed the minimum factor of safety of 1.5 for static conditions and 1.1 for seismic conditions.

Overturning Stability

Overturning of the reinforced wall was checked as follow:

$$FS_{OT} = \frac{\sum resistingmoment(M_R)}{\sum overturningmoment(M_o)}$$

Based on the anticipated subsurface conditions and the proposed MSE wall heights of 26 feet at Pier 1 and 39 feet at Pier 5, the computed factor of safety against overturning is 4.0 for static conditions and 2.0 for seismic conditions. The calculated values are greater than the required minimum values.

Sliding Stability

Sliding stability was checked at two elevations. The first is at the interface between the foundation soil and the reinforced fill. The second elevation is at the interface of the lowest geogrid and the reinforced soil. Sliding stability was checked with the following:

$$FS_{sliding} = \frac{\sum horizontalresistingforces}{\sum horizontaldrivingforces}$$

The maximum retained height of 39 feet was used for sliding stability check. The factor of safety against sliding under static conditions is 2.5 at the foundation soil and the reinforced fill and 2.0 at the lowest geogrid level. The factor of safety against sliding under seismic condition is 1.5. The calculated values are greater than the required minimum values.

Settlement

Conventional settlement analyses were conducted to confirm that total and differential settlements do not exceed the performance requirements of the project as specified in the GDM. In general, total settlement is defined as the amount of wall settlements that will occur in its design life and includes both short and long term settlements. Short term settlements occur rapidly during wall construction and may contain elastic deformation with no change in water

content. Long term settlements occur slowly according to the permeability of the soil and may continue after the completion of the wall.

Settlements from the additional loads impose by the MSE walls at Pier 1 Left and Pier 1 Right would take place almost immediately due to the absence of clays in the underlying soil. Long term total should be less than about 1 inch.

Settlements from the additional loads impose by the MSE walls at Pier 5 Left and Pier 5 Right would most likely occur within one or two months after the wall construction due to the presence of the clay. The east end of the retaining wall height is roughly 24 feet and anticipated settlement is approximately 3 inches. The west end of the retaining wall height is roughly 39 feet and anticipated settlement is approximately 4 inches. Wall settlement should be monitored during construction, and pavement construction delayed until the Engineer determines that settlement is essentially complete.

Although we do not anticipate settlements to adversely affect the performance of the new roadway, the proposed MSE walls must be able to accommodate the differential settlement. The magnitude of differential settlements will depend in part on the variations in wall height and local variations in soil conditions. The design should accommodate 2 to 4 inches of differential settlement for every 5-foot change in wall height along the length of the wall.

Post-construction facing distress such as cracking, gaps, or bulging, can occur due to excessive long term differential settlement. To minimize this potential, it is recommended that the walls be designed and constructed as two phase walls wherein the facing is attached after the majority of settlement has occurred. From a geotechnical standpoint, either a cast-in-place facing (such as shown on the Standard Plans) or precast panel facing (used on several proprietary systems) is acceptable.

INTERNAL STABILITY

The internal stability of Standard Plan MSE wall designs have been approved by WSDOT per GDM 15.5.1. Internal stability of pre-approved proprietary MSE walls is the responsibility of the Contractor. Design documentation of pre-approved systems must be submitted to the Engineer for review in accordance with the GDM (in particular, see Appendix 15-A).

DRAINAGE

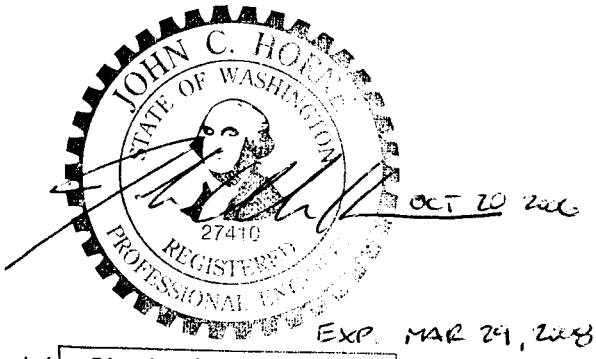
The retaining walls should be designed with permanent drainage to preclude the build-up of hydrostatic pressures and seepage forces into the reinforced soils. Collected water should be routed away from the wall to reduce the possibility of disturbing the soils at the base. We recommended providing a layer of drain rock at the back of the fill to ensure proper drainage behind the wall. The water should then be conveyed to a common discharge outlet via a culvert. In general, wall drainage shall be designed in accordance with the WSDOT GDM.

CONSTRUCTION CONSIDERATIONS

The MSE walls should be constructed in accordance with Publication No. FHWA-NHI-00-043. A minimum stripping depth of 1 foot is recommended beneath the MSE wall. Previous construction activities and other land uses may have resulted in areas of soft, compressible clayey soils, buried organic material as well as other debris. Any soft, compressible soils should be sub-excavated to firm ground under the direction of the Engineer. As a precautionary measure, it is recommended that the stripped surfaces be proof-rolled with heavy construction equipment to detect soft spots that could be indicative of soft, compressible materials. Any soft spots detected should be removed and backfilled in accordance with the GDM.

Settlement monitoring will be required to provide guidance as to when the concrete facing can be constructed for these 2 phase walls. Instrumentation should consist of survey targets established as low on the wall as practical so as to monitor as much settlement as possible, but in any case they should be deployed before the wall has achieved a height of 2 feet above existing grade. The targets should be firmly attached to the wall in such a way to limit the possibility of incidental damage due to construction activity. Two survey targets per wall are recommended – one located roughly at maximum wall height and another about 40 feet away (8 total). Elevation readings should occur immediately after target installation and at weekly intervals thereafter. Readings should be conveyed to the Engineer for review and interpretation.

Prepared by:



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References Cited:

[1] Washington State Department of Transportation, "SR-5 SR-502 Vicinity Interchange, XL-2113, Gee Creek Interchange Project, Final Geotechnical Recommendations," December 14, 2005

[2] Washington State Department of Transportation, "I-5 / SR 502 Interchange, SR 502 Over I-5 Bridge No. 5/30, Retaining Wall Layouts, Plans, and Profiles," 4 sheets, May 23, 2006

[3] Washington State Department of Transportation, "Geotechnical Design Manual," M 46-03, September 2005.

[4] American Association of State Highway and Transportation Officials, "AASHTO LRFD Bridge Design Specifications," ISBN 1-56051-250-4, 2004

[5] Federal Highway Administration, "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Design and Construction Guidelines," Publication No. FHWA-NHI-00-043, Washington, D.C., 2001.