April 3, 2009
Project No. 400943008

Mr. James A. Blomquist, Associate Vice Chancellor
City College of San Francisco
Office of Facilities Management
50 Phelan Avenue, S142
San Francisco, California 94112

Subject: Supplemental Recommendations for Tieback and MSE Walls
Ocean Campus Soccer Field, City College of San Francisco
50 Phelan Avenue, San Francisco, California

Dear Mr. Blomquist:

We understand that a mechanically stabilized, segmental block wall has been proposed to support a portion of the fill to be placed for the soccer field and a row of tieback ground anchors have been proposed to reduce the embedment depth and bending moments for the soldier-pile-and-lagging walls at the cut slope north of the soccer field. As requested, we have prepared the following recommendations for the design and construction of mechanically stabilized, segmental block walls and anchored soldier-pile-and-lagging walls on the subject project. These recommendations supplement the recommendations in our report (Ninyo & Moore, 2008) for reinforced concrete cantilever gravity walls and cantilever soldier-pile-and-lagging walls.

**MSE BLOCK WALL RECOMMENDATIONS**
Mechanically stabilized earth (MSE) retaining walls constructed of segmental, concrete masonry unit (CMU) blocks with polyethylene geogrid soil reinforcement are a suitable method to support the fill at along the southern portion of the eastern wall of the soccer field. We understand that the proposed configuration may include terraced walls with a sloping intermediate backfill and a combined grade differential of up to 12 feet. The following recommendations should be incorporated into the design and construction of these walls.
The reinforced soil zone should consist of a select granular soil that conforms with the specifications in the following table. We anticipate that significant portions of the material that will be cut from the slope north of the field will not conform with the specifications and that import fill will be needed.

<table>
<thead>
<tr>
<th>Material</th>
<th>Specifications</th>
<th>Design Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reinforced Soil (Import or on-site)</td>
<td>100% passes 2” sieve; 20% to 80% passes No. 4 sieve; 0% to 30% passes No. 40 sieve; 0% to 15% passes No. 200 sieve; pH of 5.5 to 9; sand equivalent of 30 or more; plasticity index of 10 or less</td>
<td>Internal Friction Angle $\phi$ (deg)</td>
</tr>
<tr>
<td>Retained Soil (General on-site fill)</td>
<td>Excluding unsuitable materials</td>
<td>27</td>
</tr>
<tr>
<td>Foundation Soil (On-Site)</td>
<td>Excluding unsuitable materials</td>
<td>30</td>
</tr>
<tr>
<td>Leveling Pad (Import)</td>
<td>coarse-grained angular sand, angular gravel, or crushed stone (3/4” or less) with 0% to 10% passing No. 200 sieve</td>
<td>Model as foundation soil</td>
</tr>
<tr>
<td>Drain Fill (Import)</td>
<td>Class 2 Permeable Material (Sec. 68-1.025 California Std. Specs)</td>
<td>Model as reinforced soil</td>
</tr>
</tbody>
</table>

Note: “deg” is degrees, “psf” is pounds per square foot, and “pcf” is pounds per cubic foot.

The MSE block walls should be designed in accordance with the guidelines of the National Concrete Masonry Association (Collin, 1997) for segmental retaining walls using the soil parameters listed in Table 1. The proposed walls should be checked for resistance to base sliding, overturning, bearing capacity failure, reinforcement pullout, tensile overstress, internal sliding, facial instability, and wall bulging. In our previous phase of work on this project (Ninyo & Moore, 2008) we evaluated and found that walls retaining up to 20 feet of soil on site had a suitable factor of safety against global instability. MSE block walls should be designed to resist the active earth pressure. Walls 10 feet or more in height should be designed to resist a seismic earth pressure in addition to the active earth pressure. Walls retaining flat ground should be designed to resist vehicular loads and temporary construction loading conditions. These
loading conditions may be modeled with a backfill surcharge. A reduced factor of safety for temporary loading conditions may be appropriate. The surcharge pressure of an upslope wall should be considered in the design of the lower terraced wall. The values listed in Table 2 for active earth pressure, seismic earth pressure, backfill surcharge, and terraced wall surcharge should be used to evaluate wall stability.

### Table 2 - MSE Block Wall Design Parameters

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>3:1 Rising</td>
<td>--</td>
<td>50</td>
<td>29\cdot H_t - 13\cdot H_t \cdot \text{ln}(X/H_b)</td>
<td>23</td>
</tr>
<tr>
<td>Level</td>
<td>120</td>
<td>45</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

[^1]: Lateral pressure due to surcharge on backfill of 240 psf.
[^2]: Equivalent fluid pressure.
[^3]: Lateral pressure due to surcharge of wall upslope where ‘H_t’ is height of upper wall in feet, ‘H_b’ is height of lower wall in feet, and ‘X’ is the lateral distance from the top of the lower wall to the upper wall in feet.
[^4]: Irregular seismic earth pressure may be modeled as an inverted equivalent fluid pressure with a resultant acting at about 60 percent of the retained soil height. The active earth pressure should be added to the seismic pressure to evaluate the earth pressure on the wall for seismic conditions. Seismic earth pressure may be neglected for short walls (less than 10 feet). Computed from model by Seed and Whitman (1970) using a horizontal ground acceleration of 0.25.

The site for the MSE block wall should be prepared in accordance with the recommendations in our report. The subgrade should be scarified to a depth of about 8 inches, conditioned to near optimum moisture, and compacted to 90 percent of the reference density. The reference density and optimum moisture should be evaluated in accordance with ASTM D 1557. The CMU blocks should be placed on a leveling pad approximately 6 inches thick in a trench of suitable depth to provide the appropriate embedment for the CMU block wall below the finish grade in front of the wall. The leveling pad should conform to the specifications in Table 1 and should extend about 6 inches laterally beyond the footprint of the CMU blocks. Recommended embedment depths and allowable bearing capacities are listed in Table 3.

Select fill conforming to the specifications in Table 1 should be placed and compacted in lifts below, between, and above the geogrid reinforcement. The select fill should be compacted to 90 percent of the reference density near the optimum moisture content as evaluated by ASTM D 1557. Drain fill conforming to the specifications in Table 1 should be placed and compacted in
lifts in a zone about 1-foot wide behind the CMU blocks. A perforated pipe (4-inch diameter schedule 40 PVC pipe or equivalent) should be placed at the base of the drain fill to collect and divert seepage to a suitable outlet away from the wall. The drain fill and the select fill in the reinforced soil zone should be capped with pavement or about 12 inches of soil from on-site. The wall backfill should be sloped away from the wall face or a concrete swale should be provided at the wall to collect runoff. A subdrain should be constructed at the heel of the reinforced soil zone against the retained soil. The subdrain should consist of select drain fill around a 4-inch diameter perforated PVC pipe. Solid PVC pipe with a positive gradient of 1 percent or more, should collect discharge from the perforated pipe and divert it to a suitable outlet outside of the fill area.

<table>
<thead>
<tr>
<th>Wall Height (feet)</th>
<th>Bearing Depth(^1) (inches)</th>
<th>Allowable Bearing Capacity(^2) (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 to 10</td>
<td>12</td>
<td>2,300</td>
</tr>
<tr>
<td>10 to 15</td>
<td>18</td>
<td>3,000</td>
</tr>
</tbody>
</table>

\(^1\) Measured from the bottom of the CMU wall to the grade in front of the wall. On 3:1 sloping ground (horizontal to vertical) or flatter, bearing depth should be measured at a lateral distance from the bottom the footing equivalent to 300 percent of the CMU wall width.

\(^2\) Allowable bearing capacity may be increased by one-third when considering loads of short duration such as wind or seismic loads. Listed value includes a factor of safety of 3.

The MSE block wall and geogrid reinforcement should be installed in accordance with the manufacturer’s specifications. In general, geogrid should be pulled taut while covering fill is placed to avoid introducing slack in the reinforcement. Tracked equipment should not be operated over the geogrid without suitable soil cover. Rubber-tired equipment operating on the geogrid may damage or displace the reinforcement while turning, accelerating, or braking. Overlapping geogrids at convex curves and corners should be separated by a layer of compacted fill about 3 inches thick or more. The geotechnical engineer should observe wall construction, backfilling, and geogrid placement, and perform field density testing to evaluate fill compaction.

**TIEBACK RECOMMENDATIONS**

We understand that some of the terraced retaining walls proposed for the north edge of the soccer field have been combined so that the retained soil height is up to approximately 19 feet. Soldier-
pile-and-lagging walls may be used to support the excavated face. A row of tieback ground anchors may be used to reduce the needed embedment depth and bending moments in the wall. The following recommendations should be incorporated into the design and construction of the anchored walls. These recommendations supplement those provided in Section 9.2.2 of our report (Ninyo & Moore, 2008) for cantilever soldier-pile-and-lagging walls.

Anchored soldier-pile-and-lagging walls should be designed by a structural engineer. Ninyo & Moore should be provided an opportunity to review the plans and calculations prepared by the structural engineer to check that the designed wall conforms with the intent of our recommendations.

Gravity-grouted, tieback anchors in straight shafts may be designed for an allowable ground/grout transfer stress of 2,000 pounds per square foot with an assumed factor of safety of 3 in the Franciscan Sheared Rocks. Tieback anchors should be designed for a bonded length of 15 to 40 feet and a horizontal spacing of 4 feet or more. Constructed anchors should be tested to 133 percent of the design load to check for capacity and load-deflection behavior. Anchor parameters consistent with these guidelines are presented in Table 4. The designer may select an alternate bond length within the recommended range for a proportional corresponding design load provided that the test load does not exceed 80 percent of the ultimate tendon strength.

<table>
<thead>
<tr>
<th>Tieback Anchor Type</th>
<th>Ultimate Tendon Strength (kips)</th>
<th>Design Bond Length (feet)</th>
<th>Assumed Shaft Diameter (inches)</th>
<th>Anchor Design Load (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-inch Bar Grade 150, ASTM A722</td>
<td>127.5</td>
<td>24</td>
<td>4</td>
<td>50</td>
</tr>
<tr>
<td>3 Strand Grade 270, ASTM A416</td>
<td>175.8</td>
<td>32</td>
<td>6</td>
<td>100</td>
</tr>
</tbody>
</table>

The anchor bonded zone should be located 15 feet or more below the ground surface and outside of the exclusion zone behind the wall. The exclusion zone is above a plane rising up at an angle of about 58 degrees from horizontal and offset a lateral distance from the bottom of the wall equivalent to 25 percent of the wall height. In addition, anchors should be designed for an
unbonded length of 10 feet or more for bar tendons and 15 feet or more for strand tendons. The anchors should be inclined at 15 to 30 degrees below horizontal and the anchor load divided into horizontal and vertical components for design.

Anchored walls should be designed in consideration of the earth pressures illustrated in Figures 1 and 2 for level and sloping backfill conditions, respectively. Walls retaining more than 12 feet should be designed to resist seismic earth pressures in combination with the apparent earth pressures. Walls retaining level ground should be designed to resist traffic and temporary construction loads modeled as a backfill surcharge. The surcharge pressure of an upslope wall should be considered in the design of the lower terraced wall. Recommended values of seismic earth pressure, backfill surcharge, and terraced wall surcharge are presented on Figures 1 and 2. The recommended pressures do not include a factor of safety. Depth of embedment for the soldier piles to meet shear and moment equilibrium at the anchorage should be increased by 20 to 40 percent for an approximate factor of safety of 1.5 to 2.0, respectively.

The soil parameters and lateral earth pressures listed in Table 3 and Table 12, respectively, from our geotechnical report (Ninyo & Moore, 2008) may be used to evaluate wall stability against overturning and sliding and the appropriate location of the tieback anchors below the top of wall to resist cantilever rotation during top-down construction. We previously evaluated the global stability of retaining walls at the north edge of the soccer field and found that walls retaining 20-foot high cuts at the toe of the slope north of the soccer field have a suitable factor of safety against global instability.

The soldier piles may be designed for an allowable side friction of 12L pounds per square foot to resist vertical loads (including the vertical component of anchor loads) where ‘L’ is the pile embedding length in feet. The allowable side friction may be increased by one-third for transient load combinations including wind, seismic, or tieback test loading. The soldier piles may be considered to have an effective width (with respect to earth pressures) of 3B where ‘B’ is the diameter of the concrete pier. The center-to-center spacing of the soldier piles should be equivalent to 3B. On level ground, the lagging should extend to 1 foot below grade at the bottom of the wall. Passive pressure should be neglected above the bottom of the lagging. The vertical distance
from the bottom of the lagging to the grade of the retained soil should be used as the wall height for design purposes.

Wall drainage recommendations and construction considerations for the soldier piles are presented in Section 9.2.2 of our geotechnical report (Ninyo & Moore, 2008). Anchor excavations may encounter cohesionless overburden soil, unstable rock, groundwater or perched seeps that may impact the stability of the drilled anchor holes. Temporary casing or drilling fluid should be used to stabilize the drilled holes as needed. Bentonite drilling mud should not be used as a stabilizing agent in uncased holes. Blocks of hard rock may be encountered in the Franciscan Sheared Rocks presenting hard drilling conditions. Anchors should be installed and grouted in accordance with the recommendations of the Post-Tensioning Institute (PTI, 1996).

The Federal Highway Administration’s Circular on Ground Anchors and Anchored Systems (Sabatini, et al., 1999) presents criteria for aggressive ground with a pH of less than 4.5 and electrical resistivity of less than 2,000 ohm-cm. Laboratory testing performed as part of our subsurface exploration indicates that the soil samples tested do not meet the criteria for aggressive ground and the sulfate exposure is negligible. However, we recommend that Class I corrosion protection consistent with the recommendations of the Post Tensioning Institute (PTI, 1996) be provided for the proposed ground anchors.

The tieback anchors should be tested during construction to evaluate the design assumptions and allowable pullout capacities. The contractor should provide the equipment and instrumentation for performing the test including a hydraulic jack and pump; stressing anchorage and jack chair; a calibrated pressure gage or load cell; and a free-standing, tripod-mounted, dial gage for measuring anchor movement. The equipment and test procedures should conform with the recommendations of the Post-Tensioning Institute (PTI, 1996). The test procedures generally consist of the following:

- Performance Tests - The anchor is cyclically loaded and unloaded in progressive increments up to the test load with the anchor deflection measured after each load step. The test load is held for 10 to 60 minutes with periodic deflection measurements to check for anchor creep before reduction to the alignment load to measure residual movement. Performance tests should be performed on the first two tieback anchors and 2 percent of the remaining anchors.
• Proof Tests - Anchors not designated for performance testing should be proof tested. Proof testing consists of incremental loading to the test load with deflection measurement. The test load is held for 10 minutes with periodic deflection measurements to check for anchor creep before reduction to the alignment load to measure residual movement.

The following acceptance criteria should be used to evaluate the results of the performance and proof testing:

• The apparent free length calculated from the elastic deflection of the anchor head should exceed 80 percent of the unbonded tendon length plus the jack length.
• The apparent free length calculated from the elastic deflection of the anchor head should not exceed the tendon length from the anchor head to the center of the bond length.
• Creep displacement should not exceed 1 mm between 1 and 10 min after the test load is applied otherwise the creep displacement should not exceed 2 mm between 6 and 60 minutes after the test load is applied.

Tested tieback anchors that meet these criteria may be loaded to the design lock-off load. Lift-off testing should be performed to check that the anchor load after lock-off is within 5 percent of the specified value. The geotechnical engineer should observe anchor drilling, tendon installation, grouting, and load testing to evaluate conformance with these recommendations.

We appreciate the opportunity to provide service on this project.

Sincerely,

NINYO & MOORE

[Signatures]

Peter C. Connolly, P.E., G.E.
Senior Engineer

Soumitra Guha, Ph.D., G.E.
Principal Engineer

PCC/SG/esj

Attachments: References
Figure 1 - Lateral Earth Pressures for Anchored Wall (Single Row, Level Fill)
Figure 2 - Lateral Earth Pressures for Anchored Wall (Single Row, Sloping Fill)

Distribution: (1) Addressee
REFERENCES


California Department of Transportation (Caltrans), 2006, Standard Specifications: dated May.


NOTES:

1. APPARENT LATERAL EARTH PRESSURE, $\sigma_{at} = 40H$ PSF.

2. ACTIVE EARTH Pressures, $\sigma_{a2} = 40H$ PSF AND $\sigma_{a3} = 40(H+D)$ PSF.
   Acting over width equivalent to 3B feet. Presumes pier spacing equivalent to 3B feet.

3. PASSIVE EARTH Pressures, $\sigma_{p1} = 3650$ PSF.
   Acting over width equivalent to 3B feet. Presumes pier spacing equivalent to 3B feet.

4. CONSTRUCTION OR TRAFFIC INDUCED SURCHARGE PRESSURE, $\sigma_{p1} = 120$ PSF.

5. TERRACED WALL SURCHARGE PRESSURE, $\sigma_w = 29H - 13H_1Ln(x/H)$ PSF.

6. SEISMIC EARTH PRESSURE RESULTANT, $P_e = \frac{1}{2} \cdot 23H^2$ POUNDS PER FOOT OF WALL LENGTH.

7. H, D, X, H_1, AND B IN FEET.
NOTES:

1. APPARENT LATERAL EARTH PRESSURE, $\sigma_{h1} = 50\text{ H PSF}$.

2. ACTIVE EARTH PRESSURES, $\sigma_{h2} = 50\text{ H PSF AND } \sigma_{h3} = 50 (H+D) \text{ PSF}$.
   ACTING OVER WIDTH EQUIVALENT TO 3B FEET, PRESUMES PER SPACING EQUIVALENT TO 3B FEET.

3. PASSIVE EARTH PRESSURES, $\sigma_{p1} = 3650 \text{ PSF}$.
   ACTING OVER WIDTH EQUIVALENT TO 3B FEET, PRESUMES PER SPACING EQUIVALENT TO 3B FEET.

4. SEISMIC EARTH PRESSURE RESULTANT, $P_e = \sqrt{2} \times 23 \text{ H}^2$ POUNDS PER FOOT OF WALL LENGTH.

5. H, D, X, H1, AND B IN FEET.