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Design and performance of an 11-m high block-faced geogrid wall

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ABSTRACT: An instrumented modular block-faced geogrid reinforced soil wall was constructed as part of a major roadway widening project near Seattle, WA, USA. The wall was 10.7 m high and 200 m long. A unique feature of this wall is that it was designed using the K-Stiffness Method, which resulted in approximately 35% less reinforcement than required using the current North American AASHTO Simplified Method. To test the accuracy of the K-Stiffness Method, a Class A prediction of wall strains was made at the time of wall design using assumed parameters for the soil and reinforcement properties. The Class A prediction was adjusted after the materials to be used in the wall were available for testing but before wall construction began. The measured strains are compared to predicted values. An explanation for the differences between measured and predicted reinforcement strains is that the facing column was less stiff than originally assumed in design. Nevertheless, predictions for reinforcement strains are judged to be in good agreement with measured values.

1 INTRODUCTION

The Washington State Department of Transportation (WSDOT) designed and supervised the construction of two geogrid reinforced, modular block-faced walls at a project located southeast of Seattle, WA, on SR-18 near Maple Valley, WA. The two walls retain the approach fill for a bridge that crosses the Cedar River. The walls were instrumented to monitor reinforcement strains and deflections, and global wall movements. One wall was completed at the time of this paper (Wall C: 10.7 m high and 200 m long), and is the subject of this study.

2 WALL DESIGN

The wall design was targeted to two competing wall systems that use the same geogrid reinforcement, but have minor differences in the modular facing blocks and geogrid-facing connection details. A unique feature of these walls is that they were designed using the K-Stiffness Method (Allen et al. 2003). This design procedure has been shown to result in less reinforcement than currently required by North American design codes using the AASHTO Simplified Method (AASHTO 2004).

Geogrid tensile strength and stiffness properties for design were developed from manufacturer-supplied tensile strength, installation damage, creep, and durability data and reviewed in accordance with WSDOT Standard Practice T925 (WSDOT 2004). These properties are summarized in Table 1, where J_{EOC} is the stiffness at 2% strain at the end of wall construction (assumed to be 1,000 hrs), T_{al} is the long-term allowable tensile strength, T_{ult} is the MARV for the ultimate tensile strength determined in accordance with ASTM D6637, and CR_u is the ultimate geogrid-facing connection strength divided by the ultimate geogrid tensile strength. The CR_u values were used to calculate the design long-term connection strength using the procedures in AASHTO (2004).

Table 1. Geogrid design properties.

Product	Designation	J_{EOC} (kN/m)	T_{al}/T_{ult} (kN/m)	CR_u
UXK1100	HDPE-1	350	15.1/54.0	0.73
UXK1400	HDPE-2	415	19.7/70.3	0.72
UXK1500	HDPE-3	660	33.0/115	0.68

Soil design properties were based on previous experience with materials that meet the soil specification limits used for reinforced soil walls in Washington state (e.g., slightly silty gravely sand). A plane strain friction angle of $\phi_{ps} = 41°$ and a unit weight $\gamma = 20.4$ kN/m^3 were selected.

Some details of the step-by-step procedures in the K-Stiffness Method for reinforced soil wall design were still under development at the time this wall was designed (e.g., the calculation of facing stiffness, and the selection of load and resistance factors to account for uncertainty). However, in general, the procedures provided in the WSDOT Geotechnical Design Manual (WSDOT 2005) were used. The facing stiffness is calculated assuming that the entire facing column is a continuous beam. The required long-term strength of the reinforcement using the K-Stiffness design method and the current AASHTO approach are shown in Figure 1.

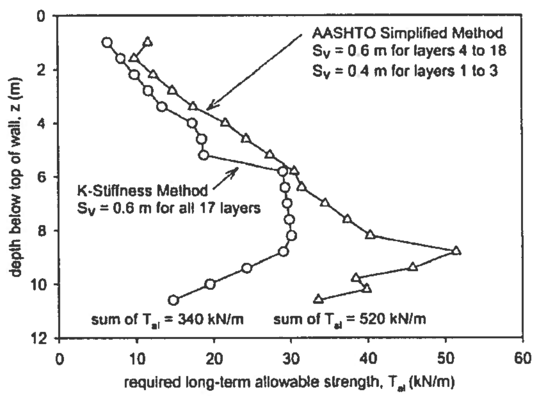

Figure 1. Summary of Wall C design.

3 INSTRUMENTATION AND INSTALLATION

Strain gauges and extensometers were mounted on four of the 16 geogrid layers in the wall. High elongation (foil-type) resistance strain gauges, Kyowa gage type KFEL-5-120-C1, were mounted in pairs at the midpoint of longitudinal rib lengths with a gauge on top and a gauge on bottom. Fiberglass rod extensometers were attached to the geogrid cross-ribs using steel plates (Figure 2) to measure geogrid deformations from which strains could be calculated.

Figure 2. Installation of extensometers.

Potentiometers were attached to the extensometer heads to record deformations. Resistance temperature devices (RTDs) were buried close to some of the strain gauges and in the wall face to monitor temperature. Temperature data was used to correct strain gauge data (i.e., influence of temperature on strain gauge leads traversing the face of the wall) and to adjust geogrid stiffness values used in design. All instruments were connected to an automated data acquisition system (data logger plus three multiplexers) with a modem for remote access and downloading of data. Readings were taken every 30 minutes during construction, and less frequently thereafter.

Figure 3 is a view of the completed wall face, showing conduits for instrumentation cables, survey targets (white spots on the wall face) for measuring lateral and vertical wall face deflections, and the data acquisition system box. The instrumentation layout for Wall C is provided in Figure 4.

Figure 3. Wall C as completed.

All but two extensometers and one RTD survived initial installation. In addition, the contractor had some difficulties maintaining the facing batter within the contract tolerances at a few locations. At these locations, the contractor used an excavator arm to push down on the soil immediately behind the facing blocks and thus move the blocks into proper alignment. This was done within 0.6 m of the third instrumented layer from the bottom, damaging some extensometers.

Since foil-type strain gauges may under-record strain (Bathurst et al. 2002), laboratory tensile tests were conducted on strain-gauged specimens to establish a correction factor (i.e. ratio of global to local strain) for each geogrid product used. A check on the correction factor was made by comparing strains calculated from the extensometer readings and strain gauge readings. From these measurements correction factors of 1.4, 1.4, and 1.2 were used to correct the strain gauge readings for the HDPE-1, HDPE-2, and HDPE-3 geogrids, respectively.

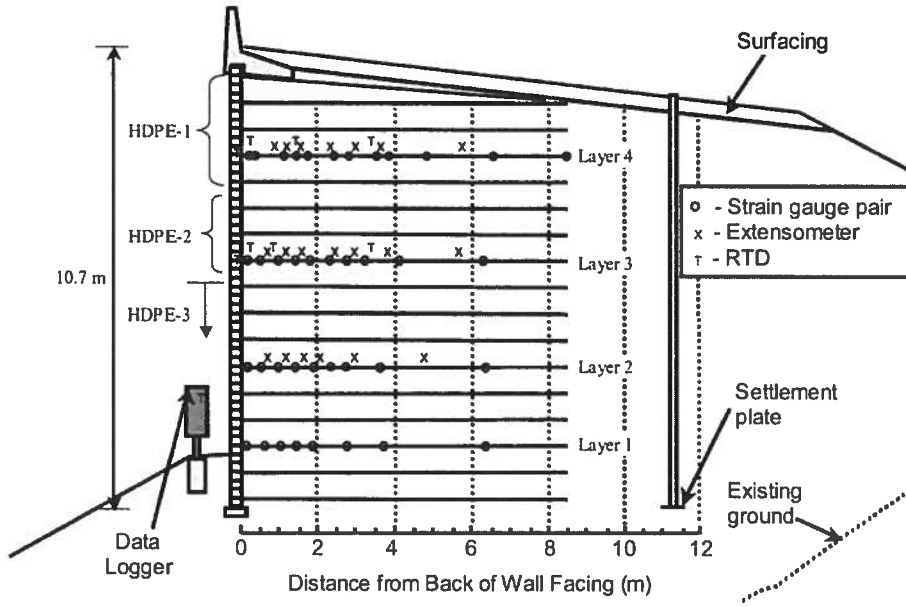

Figure 4. Cross-section of Wall C showing instrumentation layout.

4 MONITORING RESULTS AND ANALYSIS

Figure 5 presents the strain gauge readings obtained along the length of each instrumented layer multiplied by the global to local strain correction factor as discussed in Section 3.

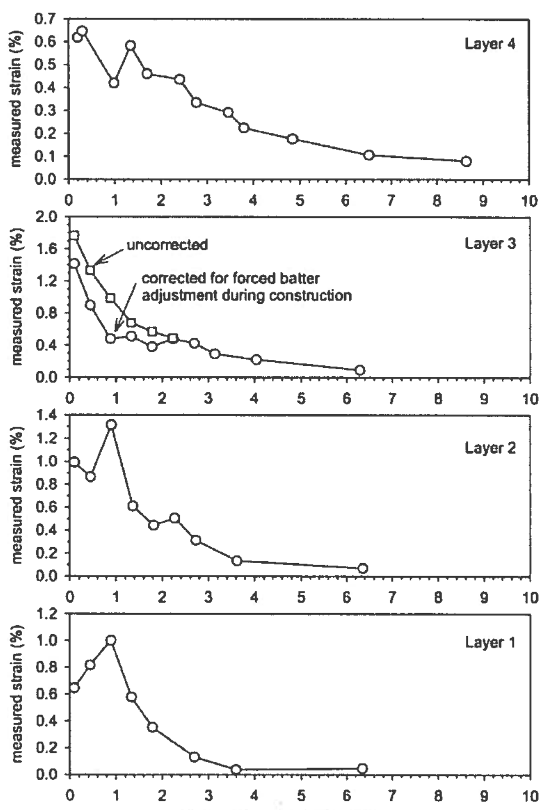

Figure 5. Measured strains in Wall C corrected to global strains.

The strains were greatest near the connection to the wall face in the upper half of the wall (Layers 3 and 4), and greatest about 1 m behind the wall face in the lower half of the wall. Figure 5 shows that one effect of the facing batter adjustment made by the contractor in the vicinity of Layer 3 was an immediate jump in strain of approximately 0.35%. Regardless of this construction event, the connection strains are quite high for this layer. Even though the strains are highest at the connection for Layers 3 and 4, there does appear to be a plateau in the strain values approximately 1.5 to 2 m behind the wall face.

Figure 6 shows the peak strains plotted as a function of height above the base of the wall. The strains are shown at the connections and for the reinforcement

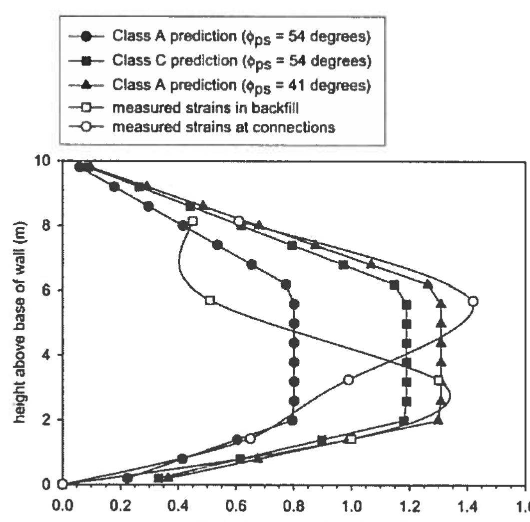

Figure 6. Predicted and measured peak strains for Wall C.

within the backfill away from the connection (i.e., the plateau areas identified in Figure 5). This plot also shows the (Class A) predicted strains using the K-Stiffness Method and design soil parameters ($\phi_{ps} = 41°$, $\gamma = 20.4$ kN/m^3) and measured soil parameters for the actual backfill material from plane strain laboratory testing ($\phi_{ps} = 54°$) and nuclear densometer testing during wall construction (γ = 22.0 kN/m^3). The figure also shows the predicted strains if the facing stiffness is adjusted to reflect field observations that there was detectable looseness in the connection and that full contact between blocks at the connection was not obtained (Class C prediction). This looseness results in the facing stiffness being controlled by a facing column segment length equal to the reinforcement spacing (0.6 m) rather than the full column height, and a reduced column thickness to account for the irregular geometry of the facing blocks. For this Class C prediction, roll-specific stiffness values were used for the geogrid, although the measured stiffness values were only slightly lower than the original design values assumed.

Figure 6 shows that the original Class A prediction made during design is very close to the strains that were measured, if not slightly conservative. When the actual soil and reinforcement properties were used instead of the design values, and the facing stiffness is based on the full column acting as a continuous member (Class A prediction), the predicted values are significantly less than the measured values. When the facing stiffness is corrected as described above (Class C prediction), the prediction is reasonably close to the measured values. Note that when connection strains are considered, the strain in Layer 3 appears to be unusually high. This is likely the result of the excessive force applied to the facing resulting from the batter adjustment method used by the contractor. These observations at Layer 3 point to the important influence of construction technique on wall performance. Another important point that arises from inspection of the strain data presented here is that the K-Stiffness Method does not (currently) explicitly address the generation of connection loads. The original development was focused on internal stability design in the vicinity of the boundary between the active and resistance soil zones. Clearly, the development of down-drag forces that can occur due to relative vertical movement between the facing and the backfill as the backfill settles and the wall facing moves outward needs to be addressed. These connection loads may be the largest loads developed in a reinforcement layer.

5 CONCLUSIONS

The retaining walls constructed as part of the Washington state SR-18 highway project are the first attempt to use the K-Stiffness Method to design full-scale field walls. This method allowed Wall C described here to be built with only 65% of the reinforcement required by current North American design codes. Performance of the wall has been excellent, in spite of the unanticipated additional reinforcement loads created by contractor efforts to adjust the facing batter. The K-Stiffness Method has been shown to give an acceptably accurate prediction of measured reinforcement strains. Based on this experience, however, improvements in the K-Stiffness Method should be made. These improvements include a more accurate assessment of facing stiffness, and design model improvements to better address the development of connection loads.

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