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Numerical modelling of two full-scale reinforced soil wrapped-face walls

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ABSTRACT

The paper reports the details of numerical models used to predict the performance of two 3.6 m-high well-instrumented wrapped-face walls. The walls were nominally identical except that the reinforcement material in one wall was a steel welded wire mesh and in the other a biaxial polypropylene geogrid. The backfill soil was modelled using both linear and nonlinear elastic-plastic constitutive models. A general hyperbolic (nonlinear) axial load-strain-time model was used for the reinforcement. The numerical results show good agreement with measured performance features for the welded wire mesh wrapped-face wall. Agreement between numerical predictions of facing displacements and maximum reinforcement loads was less accurate for the very flexible geogrid wrapped-face wall. The discrepancies are believed to be related to the unusually flexible wrapped face used in the geogrid wall construction. Numerically predicted and measured maximum reinforcement loads are compared to loads using the AASHTO reinforcement strength-based design approach (Simplified Method) and the Simplified Stiffness Method which is an empirical reinforcement stiffness-based method. The paper provides physical test data that can be used to benchmark other numerical models, highlights lessons learned during the development of the models, and identifies reasonable expectations for numerical model accuracy for models of similar complexity used to simulate the performance of mechanically stabilized earth (MSE) wall structures under operational conditions.

1. Introduction

Mechanically stabilized earth (MSE) walls are widely used for the construction of vertical and near-vertical earth retaining structures. MSE walls can be constructed with hard facings, such as modular blocks, incremental and full-height concrete panels, or with less rigid wrapped-face arrangements. Wrapped-face structures can be constructed using steel grids and welded wire mesh, or polymeric geogrid and geotextile reinforcement materials. In some cases the facing may be a wire mesh material and the soil reinforcement is a geosynthetic product (e.g., Carrubba et al., 1999). Wrapped-face walls have been used for both temporary and permanent earth retaining wall structures. Geotextile wrapped-face walls have also been used for the reinforced soil zone in two-stage false-facia wall systems (Bathurst, 2014).

The number of full-scale high-quality instrumented wrapped-face walls reported in the literature is small. Examples of geosynthetic reinforced walls are the 12.6 m-high Rainier Avenue wall in Seattle described by Allen et al. (1992), and two 3.6 m-high test walls reported by Santos et al. (2014). The only example of a well-instrumented and monitored steel wire mesh wall constructed in the field is the 17 m-high wall reported by Anderson et al. (1987). Hence, opportunities to better understand the mechanical performance of these types of walls are limited as are opportunities to compare performance features with predictions using current design and analysis methods such as force-based limit-equilibrium methods (e.g., AASHTO, 2014) and empirical-based reinforcement stiffness methods (e.g., Allen et al., 2003, 2004; Bathurst et al., 2008; Allen and Bathurst, 2015).

Numerical modelling of MSE walls with hard facings using the finite element method (FEM) has been reported in the literature by Bathurst et al. (1992), Cai and Bathurst (1995), Karpurapu and Bathurst (1995), Rowe and Ho (1997), Rowe and Skinner (2001), Yoo et al. (2011), Damians et al. (2013, 2015, 2016) and Yu et al. (2015b), amongst many others. The finite difference method

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(FDM) has been used to model MSE walls with hard and wrapped facings by Hatami and Bathurst (2005, 2006), Hatami et al. (2008), Huang et al. (2009, 2010), Bhattacharjee and Krishna (2012), Krishna and Latha (2012), Damians et al. (2014) and Yu et al. (2015a, 2015b, 2016a, 2016b). Both FEM and FDM models have been used to predict the performance of some monitored hard-face field walls to acceptable accuracy by Damians et al. (2015) and Yu et al. (2015a, 2016a). The prediction of wrapped-face walls using numerical approaches is much more limited. Zornberg and Mitchell (1994) carried out FEM modelling of the Rainier Avenue geotextile wrapped-face wall mentioned earlier. Hatami et al. (2008) conducted a numerical parametric analysis of the influence of soil type and compaction on the performance of wrapped-face walls using a FDM model. Their numerical model results were compared to the measured data for the facing profile and reinforcement strains from the laboratory full-scale welded wire mesh (WWM) wall in the current study described later. Scotland et al. (2016) carried out a series of numerical analyses that focused on the influence of construction method on geogrid wrapped-face wall deformations.

The primary objective of the current study was to develop numerical models to predict the performance of two full-scale 3.6-m high instrumented wrapped-face walls constructed at the Royal Military College of Canada (RMCC). The two walls in the current study were part of a larger study involving 11 similar walls that were constructed and instrumented to investigate and isolate the influence of facing type, facing batter, reinforcement type, properties and spacing on wall performance at the end of construction (Bathurst et al., 2000, 2006, 2009).

The two walls in the current study were nominally identical but constructed with a relatively extensible geogrid in one case and a relatively stiff welded wire mesh in the other. It is important to emphasize that selection of a PP geogrid for one wall and a welded wire mesh material for the other was not undertaken to compare the performance of different wall types based on classification into polymeric and steel reinforced MSE categories (e.g., AASHTO, 2014). Rather, the welded wire mesh was purposely manufactured to have a similar tensile strength to the PP geogrid used in one of the other reinforced soil walls in the RMCC test wall series (Bathurst et al., 2009), but with much greater stiffness. In the current investigation the axial stiffness of the welded wire reinforcement was about 30 times greater than the stiffness of the PP geogrid as shown later in the paper. Hence, differences in wall performance could be attributed to the influence of the magnitude of axial stiffness of the reinforcement when all other wall properties and geometry were nominally identical.

The numerical models were written and executed using the FDM program FLAC (Itasca, 2011). Numerical results are compared to measured performance features such as wall facing deformations, foundation pressures, and reinforcement strains and loads.

Finally, measured and numerically predicted maximum reinforcement loads are compared to values predicted using the classical limit equilibrium-based tieback wedge method (e.g., AASHTO, 2014) and the reinforcement stiffness-based method proposed by Allen and Bathurst (2015).

2. Physical test wall models

The first wall in the 11-wall test series described earlier was a reference (control) structure constructed to a height of 3.6 m with a modular dry-cast block facing and six layers of PP geogrid reinforcement. All subsequent walls in the test series were similar to the control structure but varied in one of the following details: wall facing type, facing batter, reinforcement type and properties, or reinforcement vertical spacing. The reason for this approach was to identify as far as practical the influence of each of these component details on the performance of otherwise nominally identical wall structures. Details of some of these walls have been described by Bathurst et al. (2000, 2006, 2009).

Two of the RMCC walls that are the focus of the current study were built using a wrapped-face with the same wall geometry (Fig. 1). However, one wall was constructed with a relatively inextensible soil reinforcement material (welded wire mesh - WWM) and the other with a relatively extensible polymeric material (biaxial polypropylene (PP) geogrid). The wire mesh was comprised of steel wire of 2 mm diameter (14 gauge) and apertures of 200 (between longitudinal wires) by 100 mm (between transverse wires). The wire mesh was specially fabricated to give a low tensile yield strength of 7 kN/m. The PP geogrid for the polymeric wrapped-face wall in the current study was a biaxial product with an index ultimate tensile strength of 14 kN/m in the machine direction, which was the direction of loading in the wall. The aperture sizes of the geogrid were 25 by 33 mm in longitudinal and transverse directions, respectively.

The walls were 3.6 m high and 3.4 m wide at the face. The reinforced soil zone plus backfill behind the reinforced walls extended 5.65 m beyond the toe of the wall to the back of the test facility. The reinforcement vertical spacing was 0.6 m.

The inside walls of the test facility were covered with a composite arrangement of plywood, Plexiglas, and lubricated polyethylene sheets to minimize the friction between the side walls and backfill soil and thus approach a plane strain test condition as far as practical. The same technique was used for all other walls in the same series of tests noted earlier.

The wrapped-face walls were constructed on a 0.1 m-thick soil levelling layer to separate the first reinforcement layer from the concrete foundation. Each facing wrap except the top layer was attached to the reinforcement layer above using a metal bar clamp (Fig. 1b). In actual wrapped-face walls in the field, the top of the facing wrap is taken back and down into the reinforced soil fill for all layers. The clamp arrangement used here was purposely adopted to facilitate comparison of the two wrapped-face walls in this paper with performance of the control wall that was built with a dry stack of modular blocks. The back of the modular block column matches the straight line drawn through the clamps at the target facing batter of $\omega = 8°$ from the vertical (Bathurst et al., 2006). The difference in wall performance between the control modular block wall in the larger experimental program and the wrapped-face wall with the same geogrid reinforcement and geometry can thus be ascribed to the effect of the facing. Bathurst et al. (2006) concluded that at end of construction, the reinforcement loads in the geogrid wrapped-face wall were up to 3.5 times greater than the loads in the matching modular block wall because a large portion of the lateral earth pressure developed in the reinforced soil mass was carried by the relatively stiff modular block facing column.

The clamp arrangement described above was adopted for the reasons just described. However, the PP geogrid wrap sections in a true field wall would be expected to be stiffer as a result of the additional length of reinforcement that is taken back into the soil fill. The consequences of this difference in construction detail are discussed later in the paper when numerical and measured reinforcement loads are compared to analytical solutions for the calculation of maximum reinforcement loads at end-of-construction (operational) conditions using two different design methods.

The two wrapped-face walls were constructed using the "moving formwork" construction technique (Holtz et al., 1997). A target facing batter of $\omega = 8°$ from vertical was maintained by the forms which were braced against the front of the test facility. During construction, two wrapped-face layers were supported

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