Lateral Load Capacity of Cast-in-Place Shafts behind an MSE Wall

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ABSTRACT

Current practice for designing laterally loaded cast-in-place shafts that pass through an MSE Wall involves isolating the shafts from the MSE mass and anchoring the shafts into the underlying foundation material. Sizeable cost and time savings could be realized, while still maintaining stability and reliability, if a method were available to evaluate the lateral load capacity of a shaft that is supported by the MSE mass alone with no rock socket.

Construction, instrumentation, and testing of multiple 0.9m (36in.) diameter shafts solely supported by a 6m (20 ft) MSE block wall was conducted for the Kansas Department of Transportation (KDOT). This paper describes the design and construction of the wall and shafts, and the results from the lateral load tests of two of the shafts. These shafts had lengths that were equal to the full height of the wall and 75 percent of the full height of the wall to evaluate the reduction in capacity if shorter foundation elements suspended in the MSE mass were used. Results for both load and deflection of the shafts and the relative deflections of the shafts and wall facing during loading are presented.

INTRODUCTION

The potential for significant savings has led to the wide use of mechanically stabilized earth (MSE) walls. These wall systems have proven to be a flexible and cost effective technology for many applications; however they do require a significant amount of space behind the wall for the reinforced mass. This presents a challenge for conditions where the foundation for a sound wall, bridge abutment, or other feature must occupy or pass through the reinforced space. There is no commonly accepted design method for MSE walls that provide lateral support to drilled shafts

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that pass through the geosynthetic reinforcement. As a result, shafts are often designed such that their lateral loads pass through and are isolated from the MSE wall and are carried by a rock socket foundation. This leads to a significantly more expensive foundation.

An experimental program devised to test the lateral load response of shafts that are supported by and contained within an MSE wall was initiated by the Kansas Department of Transportation (KDOT) in cooperation with the University of Kansas (KU). This paper contains a discussion of two of the shafts tested. This type of design provides substantial savings in cost and time to designers and contractors. Each test shaft was 0.9m (3 ft) in diameter and the distance from the back of the wall facing to the center of the shaft was 1.8m (6ft). Each shaft was centered within a test section whose total width was 4.6 m (15 ft). The depth of one shaft was equal to the full 6m (20 ft.) height of the wall (Shaft B), and the depth of the second shaft was 75% of the wall height or 4.6 m (15 ft) (Shaft BS). Both shafts were laterally loaded in the direction of the wall facing to failure.

CONSTRUCTION AND INSTRUMENTATION

The test site was located in Wyandotte County, Kansas and was selected based on the presence of a limestone base to eliminate settlement as an issue. The wall was designed and constructed in accordance with FHWA procedures for MSE walls without shafts (FHWA, 1997). At each shaft location a 0.9m (3 ft) diameter corrugated metal pipe (CMP) was placed to act as a form for the concrete and to prevent aggregate from entering the shaft area. A concrete leveling pad was cast to provide vertical alignment for the modular block facing. Blocks were placed, aligned and leveled to provide appropriate wall batter and alignment. The reinforcement layers consisted of uniaxial high density polyethylene punched-drawn geogrid with an ultimate tensile strength of 114 kN/m (7810 lb/ft) for the lower reinforcement layers and an ultimate tensile strength of (4800 lb/ft) for the upper layers when tested in accordance with ASTM D 6637 (layers referred to as G1 and G2 in this paper). Reinforcement was spaced vertically every 0.6m (2 ft) of elevation. The lower four layers consisted of G1 and the upper six layers consisted of G2. The geogrid was cut to fit around the CMP. All slack was removed from the geogrid before backfill was placed over it. Backfill material consisted of a clean crushed limestone rock whose specifications were established by KDOT as CA-5. The CA-5 specification consists of 19mm (3/4 in.) maximum size and more then 95% of particles retained on the U.S. #8 sieve. The CA-5 used in the project had a peak friction angle of 51° based on large diameter triaxial cell testing for confining stresses within the range of the wall (34.5 – 138 kPa [5-20 psi]). Compaction of the aggregate consisted of several passes of a large steel wheeled roller, where possible, and a plate compactor near the face of the wall. A 20 cm (8 in.) low permeability cover was placed above the aggregate fill. Vertical slip joints were located between test sections in an attempt to isolate the test sections from each other. For each slip joint the geogrid and facing blocks were cut such that forces could only be transmitted across the slip joint through aggregate interlock.

After the wall was constructed, the steel reinforcement cages were lowered into the CMP forms. The cages consisted of 12 evenly spaced #11 bars for longitudinal reinforcement and #5 hoops for transverse reinforcement spaced every 152 mm (6 in.) for the first 0.9 m (3 ft.) and every 305 mm (1 ft.) for the remainder. High slump (229 mm or 9 in.) concrete was poured having an average compressive strength of 45 MPa (6,500 psi).

Instrumentation consisted of three systems (Figure 1). Monitoring of the shaft top was done using five LVDTs, a hydraulic pressure gage, and a load cell attached to a data acquisition system. Each test shaft and reaction shaft had two LVDTs attached and the hydraulic ram also had an LVDT to serve as a check of the shaft LVDTs. The hydraulic pressure gage served as a check for the load cell. Inclinometers were used as a second check of the LVDTs and to determine the magnitude of any of shaft bending. A second data acquisition system was used to monitor performance of the MSE wall using earth pressure cells and strain gages. Movement of telltales installed within the fill and attached to the geogrid, as well as targets attached to the wall facing, was monitored using a digital camera and a photogrammetric process. Additional details on construction and instrumentation are located in Pierson (2008, 2009).

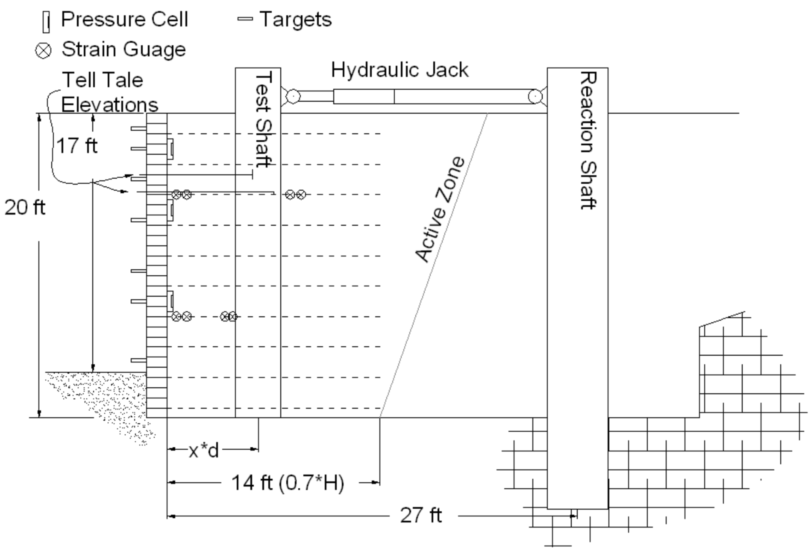

FIG. 1 Sketch of Shaft B and instrumentation locations.

The photogrammetric process used to monitor movements of the wall facing and telltales consisted of 84 PVC targets attached to the wall with a 0.152m (6 in) scale on each target, and telltales which passed through notches in the wall facing. Images of the wall, including the targets and telltales, were taken using a high-quality digital SLR camera before and during each test. These images were then downloaded into AutoCAD and used to determine wall or telltale displacement at each measurement location. This method permitted essentially instantaneous measurement of displacements at multiple wall locations throughout the test, although there was a significant amount of time required for post processing. A more detailed discussion of this use of photogrammetry is discussed in Pierson (2009).

RESULTS

Testing was displacement controlled. Hydraulic pressure was increased and movement was initiated until the desired displacement was achieved. The hydraulic cylinder valves were then closed to prevent any further cylinder movement and remained closed for the greater of 5 minutes or until the inclinometer measurements were completed. A small amount of shaft deflection and decrease in cylinder load occurred during the holding time as the wall/shaft system adjusted to the new loading conditions. Therefore three values of load and deflection were reported; a peak value for each step, a value at 2.5 minutes after the peak, and the final load before initiation of the following step (Figures 2a and 2b). A summary of loads vs. deflections is reported in Table 1. For this table and for Table 2 the distance from facing refers to the distance from the back of the facing to the center of the shaft.

Lateral load capacity was substantial with loads in excess of 200 kN recorded for a shaft displacement of 25mm for Shaft B. Loads for a given displacement for Shaft BS were somewhat lower. For example, at 20 mm of displacement Shaft BS achieved 65% of Shaft B's load, and at 140 mm of displacement Shaft BS achieved 62% of Shaft B's load. A preliminary p-y curve comparison suggests the pile capacity was approximately 15% of a similar shaft in the crushed stone with no wall.

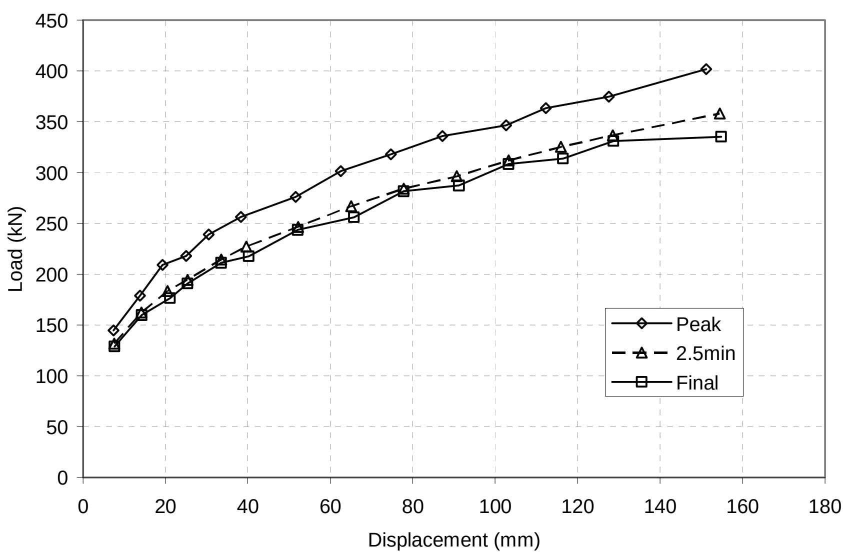

FIG. 2a. Shaft B peak, 2.5 minute, and final load versus displacement.

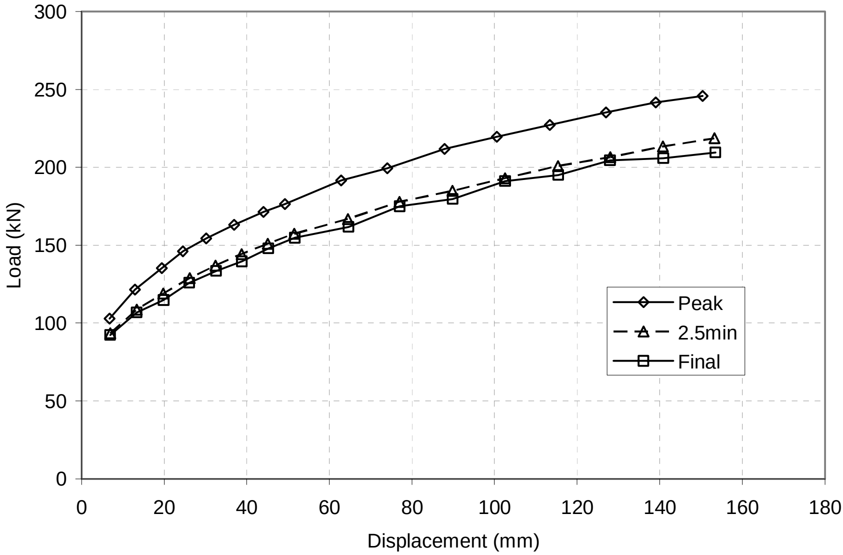

FIG. 2b. Shaft BS peak, 2.5 minute and final load versus displacement.

Table 1. Load vs. Shaft Displacement

Shaft	Distance From Facing (m)	Displacement	Peak Load for a given displacement (kN)					
			12mm	25mm	50mm	75mm	100mm	Ultimate
B	0.9		160	220	270	310	340	400
BS	0.9		120	147	178	199	219	246
			Final Load for a given displacement (kN)					
B	0.9		140	196	245	282	310	334
BS	0.9		107	125	156	174	182	209

Wall facing displacement was measured using photogrammetry with a row of targets on the wall at an elevation of 5.4m and vertical columns of targets on the wall in front of each shaft. Wall displacements for a given load are reported in Table 2. Figure 3 shows displacement of the wall facing at an elevation of 5.4m for a given displacement of Shaft B. Displacements at this elevation for Shaft BS were similar. Wall displacements were significantly less than shaft displacements for a given load. The full depth shaft (Shaft B) had a width of influence of 2.7m on either side of the centerline of loading, where the width of influence is defined as a displacement equal to 10% of the maximum wall displacement. While the shapes of wall displacement at elevation 5.4m was similar near the top of the wall for Shafts B and BS, shaft BS caused significantly less displacement of the wall at lower elevations as shown in Figures 4a and 4b.

Table 2. Load vs. Maximum Wall Facing Displacement

Shaft	Distance From Facing (m)	Max Wall Displacement	Peak Load for a given displacement (kN)					
			12mm	25mm	50mm	75mm	100mm	Ultimate
B	0.9		214	258	311	350	387	400
BS	0.9		125	156	191	218	236	245
			Final Load for a given displacement (kN)					
B	0.9		191	222	271	311	334	338
BS	0.9		106	138	165	190	205	209

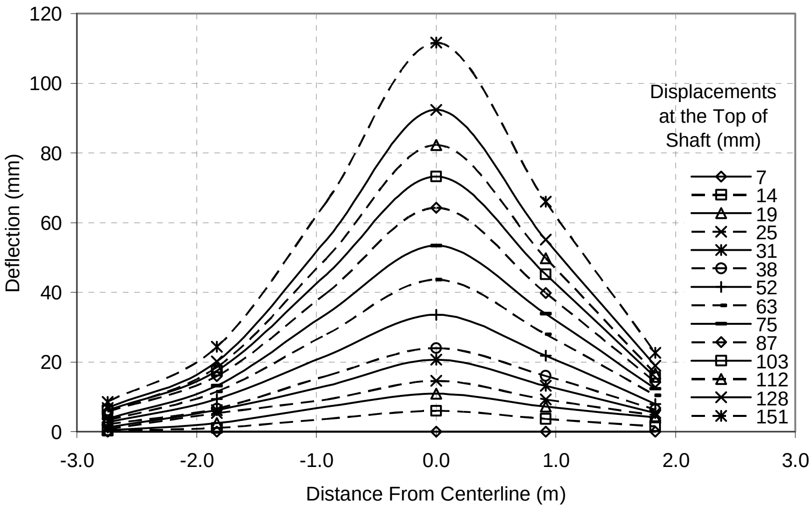

FIG. 3. Plan view of Shaft B wall facing displacement at El. 5.4 m.

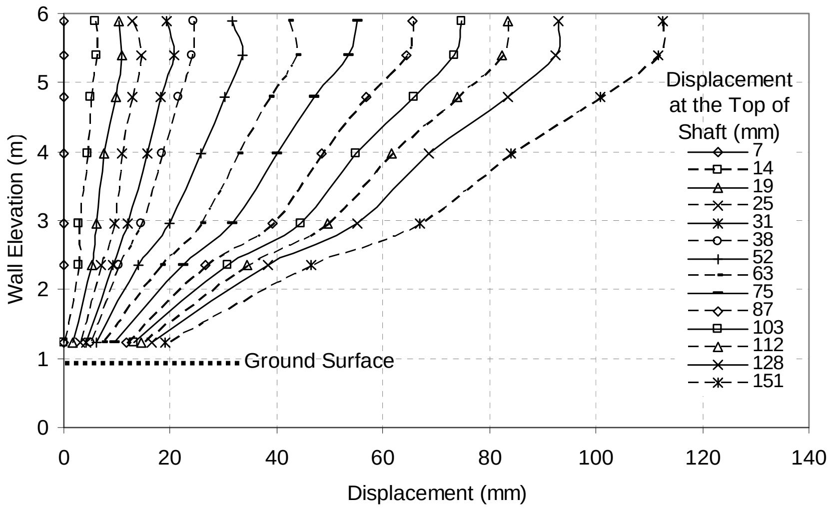

FIG. 4a. Wall facing displacement for Shaft B.

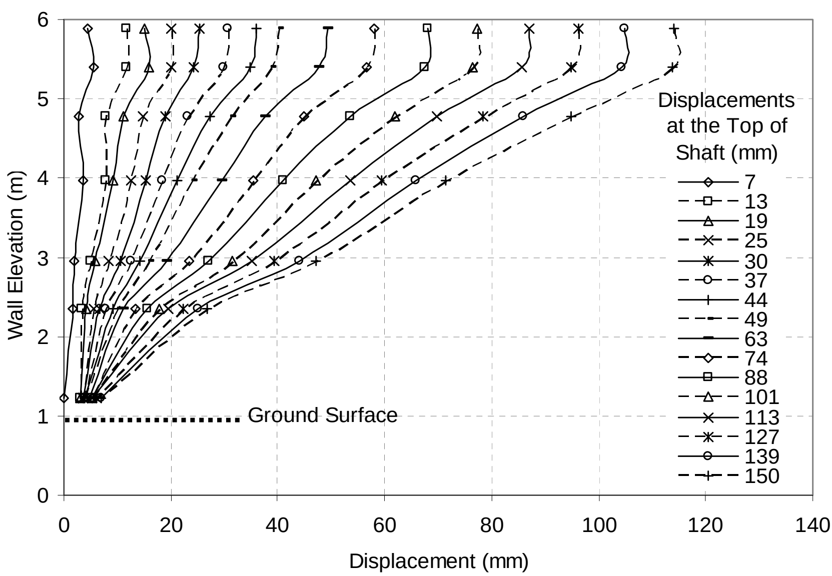

FIG. 4b. Wall facing displacement for Shaft BS.

GENERAL OBSERVATIONS

Although significant displacements of the wall occurred, the masonry block surface texture concealed the deformations well such that they were difficult to see from in front of the wall without close inspection, even for deformations in excess of 100mm.

CONCLUSIONS

A full scale MSE wall was constructed with drilled shafts contained within the reinforced backfill. The shafts were loaded laterally in the direction of the wall and the following conclusions were reached:

- Full depth shafts installed within two diameters of the back of an MSE wall can support lateral loads in excess of 150 kN with limited deformations, and had a total capacity in excess of 300 kN.
- Shorter shafts founded within the fill have substantially less capacity than their full depth counterparts; but can still carry substantial lateral loads.
- The width of influence of both shafts on the wall was approximately 2.7 meters on either side of the shaft centerline and deformations were greatest near the top of the wall.
- The shorter shaft did not have as great of an impact on the lower portion of the wall facing as the full depth shaft.

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