JOINT TRANSPORTATION RESEARCH PROGRAM

INDIANA DEPARTMENT OF TRANSPORTATION AND PURDUE UNIVERSITY



Performance Assessment of MSE Abutment Walls in Indiana



Venkata Abhishek Sakleshpur, Monica Prezzi, Rodrigo Salgado

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JOINT TRANSPORTATION RESEARCH PROGRAM

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EXECUTIVE SUMMARY

PERFORMANCE ASSESSMENT OF MSE ABUTMENT WALLS IN INDIANA

Introduction

This report presents a numerical investigation of the behavior of steel strip-reinforced mechanically stabilized earth (MSE) direct bridge abutments under static loading. Finite element simulations were performed using an advanced two-surface bounding plasticity model based on critical state soil mechanics. Results of the simulations were found to be in good agreement with published laboratory and field measurements, including horizontal-facing displacements and tensile forces in the reinforcement.

A parametric study was then conducted to investigate the behavior of a full-scale direct MSE bridge abutment. The parameters considered were the horizontal distance of the footing behind the wall facing, backfill compaction, reinforcement length and spacing, and magnitude of bridge load. Results indicate that these parameters have a significant influence on the horizontal-facing displacements, bridge footing settlements, and axial strains in the reinforcements.

A survey questionnaire on the current state-of-practice of direct and mixed MSE abutments was prepared and distributed to all departments of transportation (DOTs) in the United States. Results obtained from the survey shed light on

- percentage of use of direct and mixed MSE abutments by various DOTs;
- abutment height, type and dimensions of the facing element;
- type of reinforcement, proportioning of footing and pile in direct and mixed MSE abutments, respectively; and
- common problems experienced by DOTs with respect to construction and performance of MSE abutments in the field.

Findings

- Results from the parametric study indicate that the horizontal distance from the back of the facing to the front edge of the footing, backfill compaction, reinforcement length and spacing, and bridge load have significant influence on the horizontalfacing displacements, bridge footing settlements, and axial strains in the reinforcements. For a given bridge load, abutment movements can be reduced by properly compacting backfill soil (especially within the 1 m distance behind the wall facing), decreasing reinforcement spacing, and increasing reinforcement length.
- Based on the results obtained from the finite element simulations performed in this study, it is recommended that the clear horizontal distance from the back of the wall facing to the front edge of the footing be within 0.15 to 0.2 times the height *H* of the wall facing measured from the ground surface to the top of the facing, with a minimum of 0.1*H*.
- The depth of embedment of the footing is suggested to be within 0.2 to 0.25 times the width of the footing. The minimum vertical clearance between the base of the footing and the top level of reinforcement should be 0.3 m. A reinforcement length of 0.7*H* is suggested as a reasonable starting point for pre-liminary design and internal stability analysis of a direct MSE abutment.

- A DOT survey was carried out to obtain information on the current state-of-practice of MSE abutments in various US states. An email solicitation was distributed to all 50 DOTs, and responses were received from 31. It was found that 83.9% of the DOTs have constructed MSE abutments in their respective states, while 16.1% reported on the contrary, and 63.9% and 69.2% of the DOTs have constructed direct and mixed MSE abutments, respectively, with heights of 21 to 30 ft.
- 50% of the DOTs use only precast concrete panels as the facing element for both direct as well as mixed MSE abutments. Steel strips are the preferred choice of reinforcement by most DOTs (used by 40% of them) for both direct and mixed MSE abutments.
- 46.4% of the DOTs reported the clear horizontal distance from the back of the wall facing to the front edge of the footing to be within 2 ft., while 39.3% of the DOTs reported it to be within 2.1 to 4.0 ft. The minimum requirement specified by FHWA (2009) is 0.5 ft.
- 37.5% of the DOTs reported the depth of embedment of the footing to be between 1.1 and 2.0 ft. However, no guidelines have been specified by FHWA (2009) for the depth of embedment of the footing in a direct MSE abutment.
- 46.2% of the DOTs reported the vertical clearance between the base of the footing and the topmost reinforcement layer to be within 0.6 to 1.0 ft. The minimum requirement specified by FHWA (2009) is 1 ft.
- 50% of the piles used by DOTs in mixed MSE abutments are partial-displacement piles, 31.25% are displacement piles and 18.75% are non-displacement piles. 78.1% and 21.9% of partial-displacement piles are H-piles and open-ended pipe piles, respectively, whereas 80% and 20% of displacement piles are closed-ended pipe piles and prestressed concrete piles, respectively. The non-displacement piles consist of drilled shafts.
- 53.8% of the DOTs reported the clear horizontal distance from the back of the wall facing to the front edge of a driven pile to be between 2.1 and 4.0 ft., while 28.2% reported it to be between 4.1 and 6.0 ft. The minimum requirement specified by FHWA (2009) is 1.5 ft.
- 61.5% of the DOTs reported the clear horizontal distance from the back of the wall facing to the front edge of a drilled shaft to be between 2.1 and 4.0 ft. while 23.1% reported it to be between 4.1 and 6.0 ft. The minimum requirement specified by FHWA (2009) is 3 ft.
- The top five problems experienced by DOTs with respect to construction and performance of MSE abutments are
 - 1. loss of backfill material through the joints of the facing (18.1%);
 - 2. inadequate drainage system (12.5%);
 - 3. unsatisfactory workmanship and QA/QC (12.5%);
 - 4. growth of vegetation in the joints of the facing (11.1%); and
 - 5. inconsistent backfill compaction (11.1%).

Implementation

Based on the results obtained from the finite element simulations performed in this study, the recommendations provided for (a) the clear horizontal distance from the back of the wall facing to the front edge of the footing, (b) the depth of embedment of the footing, and (c) the length of the reinforcement can be taken into account during design and construction of direct MSE abutments in Indiana.

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1. INTRODUCTION

1.1 Background

Retaining structures, such as conventional gravity and cantilever retaining walls, mechanically stabilized earth (MSE) walls and abutments, and deep excavation diaphragm walls, constitute a vital part of highway infrastructure projects that include flyovers, bridges and underground metros. In the past, conventional retaining walls (gravity or cantilever) were often the first choice. However, with the advent of soil reinforcement towards the latter half of the previous century, MSE walls have often been selected for transportation infrastructure projects in the United States because of the many advantages they offer when compared to conventional walls. In fact, the concept of earth reinforcement was proposed centuries ago and has since been used in many different ways to enhance the behavior of weak materials. Natural reinforcements, such as straws, sticks and branches, were used to reinforce natural building materials in order to produce a stiffer composite material.

Soil reinforcement technology, first pioneered by Henri Vidal in the 1960s, has been used in the U.S. since the 1970s in various geotechnical applications, such as reinforced soil retaining walls, reinforced slopes/ embankments, reinforced foundation beds, and reinforced pavements. MSE walls are flexible retaining systems composed of facing units, backfill, and reinforcement. The facing units typically consist of either precast concrete panels or modular blocks while the backfill material is ideally coarse-grained in nature. The reinforcement may be either inextensible (metallic strips or grids) or extensible (geosynthetic) and is connected to the wall facing units (Jones, 1996). MSE walls accommodate larger differential and post-construction settlements than their traditional counterparts, due to their ability to distribute deformations. As illustrated in Figure 1.1(a) and (b), there are basically two types of MSE bridge abutment walls: (1) mixed MSE abutment (MSE wall combined with pile foundations) and (2) direct MSE abutment.

A mixed MSE abutment is a pile-supported abutment. The MSE wall only provides lateral support for the approach embankment, whereas the piles support mainly the loads from the bridge (Figure 1.1(a)). Piles are either driven through the constructed MSE wall or installed before construction of the MSE wall (Berg & Vulova, 2007; Brabant, 2001). In the direct MSE abutment design (Figure 1.1(b)), the bridge abutment seat sits directly on a footing, which is built on top of the MSE wall. The MSE wall retains the approach embankment and supports the loads from the bridge (Berg & Vulova, 2007; Zevgolis & Bourdeau, 2007).

Although the response of conventional MSE walls has been well investigated, both experimentally (Allen & Bathurst, 2014; Bathurst, Walters, Vlachopoulos, Burgess, & Allen, 2000; Bathurst et al., 2009; Runser, Fox, & Bourdeau, 2001; Stuedlein, Bailey, Lindquist, Sankey, & Nelly, 2010) as well as numerically



(b)

Figure 1.1 Schematic illustration of MSE abutments: (a) mixed and (b) direct.

(Abdelouhab, Dias, & Freitag, 2011; Cristelo, Felix, Lopes, & Dias, 2016; Damians, Bathurst, Josa, & Lloret, 2015; Huang, Bathurst, & Hatami, 2009; Ling & Liu, 2009; Yu, Bathurst, & Allen, 2016; Yu, Bathurst, & Miyata, 2015), comparatively less work has been conducted to study the behavior of MSE walls used as abutments for bridge support. Numerical analyses (finite element and finite difference analyses) of the behavior of MSE abutments (Helwany, Wu, & Kitsabunnarat, 2007; Skinner & Rowe, 2005; Zheng & Fox, 2016) have employed relatively simple constitutive models for the backfill, such as Mohr-Coulomb with non-associated flow rule and Duncan-Chang hyperbolic relationship, and cap plasticity model with Drucker-Prager failure criterion. In this study, finite element simulations of direct MSE abutments are performed using an advanced two-surface bounding plasticity model based on critical state soil mechanics.

1.2 Organization of Report

The report has been organized into 7 chapters. Chapter 1 provides the introduction and background to the study. Chapter 2 focuses on the literature review, detailing the existing guidelines for the construction of direct and mixed MSE abutments, case histories and field load tests of MSE abutments. Chapter 3 explains the finite element (FE) model used for simulating the response of the MSE abutments. Chapter 4 presents the results obtained from the FE simulations while Chapter 5 validates the predictions with measured data of instrumented MSE abutments in the literature. Chapter 6 documents the responses received from the state-ofpractice survey of MSE abutments provided by various Departments of Transportation in the United States. Chapter 7 presents a summary of the entire work with conclusions and recommendations for future research.

2. LITERATURE REVIEW

2.1 Ratio of Coefficient of Lateral Earth Pressures K_r/K_a

Research studies on MSE walls have indicated that the maximum tensile force in the reinforcement is primarily related to the type of reinforcement, which in turn is a function of the modulus, extensibility and density of reinforcement (Allen, Christopher, Elias, & DiMaggio, 2001; Christopher et al., 1990; Collin, 1986). Figure 2.1 shows the ratio K_r/K_a of the lateral earth pressure coefficient to the Rankine active earth pressure coefficient to be used in the design of an MSE wall versus depth below the top of the wall facing for different types of reinforcement. The K_r/K_a ratio for metallic reinforcement decreases from values of 1.7 (for metal strips) and 2.5 (for metal bar mats and welded wire grids) from the top of the wall facing to a value of 1.2 at a depth of 6.1 m (20 ft) below the top of the facing and remains constant thereafter. On the other hand, the K_r/K_a ratio for geosynthetic sheet reinforcement is independent of the depth below the top of the wall facing and is equal to a value of 1.0.

2.2 Pullout Resistance of Reinforcement

The pullout resistance of reinforcement is defined by the ultimate tensile load required to generate outward sliding of the reinforcement through the reinforced soil mass. Several approaches and design equations have been developed and are currently used to estimate the pullout resistance of reinforcement by considering frictional resistance, passive resistance, or a combination of both. According to FHWA (2009), the pullout resistance of reinforcement can be calculated as:

$$P_{ult} = F^* \alpha \sigma'_v L_e B C \tag{2.1}$$

where

 P_{ult} = pullout resistance of reinforcement

 F^* = pullout resistance factor

 α = scale effect correction factor to account for nonlinear stress reduction over the embedded length of the reinforcement (equal to 1.0 for metallic reinforcement, 0.8 for geogrids and 0.6 for geotextiles)

 σ'_{ν} = vertical effective stress at the depth of the reinforcement-soil interface



Figure 2.1 K_r/K_a vs. depth below top of wall facing for different types of reinforcement (modified after Elias & Christopher, 1997).

 L_e = embedment length of reinforcement in the zone of resistance beyond the slip surface

B = width of reinforcement

C = effective unit perimeter of the reinforcement (equal to 2.0 for sheet, strip and grid reinforcements)

 $L_e \times C =$ total surface area per unit width of reinforcement in the zone of resistance beyond the slip surface

In the absence of site-specific pullout test data, FHWA (2009) suggests that the following semi-empirical relationships should be used in conjunction with the standard specifications for a reinforced backfill to provide a conservative estimate of the pullout resistance of the reinforcement. For ribbed steel strips, the pullout resistance factor F^* can be estimated as (FHWA, 2009):

 $F^* = \tan \rho = 1.2 + \log C_u$ at the top of the wall facing

$$= 2.0 \text{ maximum}$$
 (2.2)

 $F^* = \tan \phi_p$ at a depth of 20 ft (6m) and below (2.3)

where ρ is the interfacial friction angle between the backfill and the reinforcement, C_u is the coefficient of uniformity of the backfill (= D_{60}/D_{10}), and ϕ_p is the peak friction angle of the backfill. If the specific value of C_u for the backfill is unknown at the time of design, FHWA (2009) recommends that a C_u value of 4 may be assumed, which corresponds to an F^* value of 1.8 at the top of the wall facing.

For geotextiles and geogrids, the pullout resistance is based on a reduction factor that operates on the peak friction angle of the backfill. In the absence of pullout test data, FHWA (2009) suggests that the value of F^* for geotextiles and geogrids may conservatively be taken as:

$$F^* = \frac{2}{3} \tan \phi_p \tag{2.4}$$

According to FHWA (2009), for MSE walls using select granular backfill, the maximum value of ϕ_p to be used in design is 34° unless project specific test data substantiates higher values. A lower bound value of 28° is suggested by FHWA (2009).

2.3 Interface Shear Between Reinforcement and Backfill

The interface shear between geosynthetic sheet reinforcement and backfill soil is often lower than the peak friction angle of the soil itself and can hence form a slip plane. According to FHWA (2009), the interface friction coefficient tan ρ should be determined from soil– geosynthetic direct shear tests in order to evaluate sliding along the geosynthetic interface with the backfill. In the absence of test results, FHWA (2009) suggests that ρ may conservatively be estimated as:

$$\rho = \frac{2}{3} \tan \phi_p \tag{2.5}$$

2.4 FHWA Guidelines for MSE Abutments (FHWA, 2009)

MSE bridge abutments have been traditionally designed to support the bridge superstructure either on a spread foundation constructed directly on the reinforced soil zone (direct MSE abutment), or on piles constructed through the reinforced soil zone (mixed MSE abutment). According to FHWA (2009), direct MSE abutments may be more economical than mixed MSE abutments and thus should be considered when the anticipated settlement of the footing and reinforced volume is rapid/small or essentially complete, prior to the construction of the bridge beams. Based on field studies, AASHTO (2012) specifies that tolerable angular distortions between abutments or between piers and abutments be limited to 0.008 radians for simple spans and 0.004 radians for continuous spans, to prevent overstressing or causing damage to the superstructure elements.

2.4.1 Direct MSE Abutment

FHWA (2009) suggests that the following important guidelines be implemented during design and construction of direct MSE abutments:

- 1. The minimum horizontal distance from the front of the wall facing to the centerline of the bridge bearing should be 3.5 ft (1 m).
- 2. The minimum horizontal distance from the back of the wall facing to the front edge of the footing should be 0.5 ft (0.15 m).
- 3. The minimum vertical clearance between the bottom of the footing and the top level of reinforcement should be 1 ft (0.3 m).
- 4. The allowable pressure on the footing should be limited to 4 ksf (200 kPa) and 7 ksf (335 kPa) (factored) from the points of view of serviceability and strength, respectively.
- 5. The maximum horizontal force at the top reinforcement level should be used for the design of facing-reinforcement connections at all the other reinforcement levels.

2.4.2 Mixed MSE Abutment

In situations where it is not possible to construct direct MSE abutments due to either unacceptable postconstruction settlements or other reasons, a mixed MSE abutment can be constructed wherein the bridge superstructure is placed on stub footings supported by deep foundations, such as driven piles or drilled shafts (Figure 1.1(a)). In this configuration, the vertical loads from the bridge deck are not considered in the analysis since they are transmitted directly to a deep and competent bearing stratum by the piles. However, the horizontal force on the wall facing is resisted by the mobilized frictional resistance between the backfill and the reinforcement.

FHWA (2009) suggests that the following important guidelines be implemented during design and construction of mixed MSE abutments:

- 1. The minimum horizontal distance from the back of the wall facing to the front edge of a driven pile should be 1.5 ft (0.5 m).
- 2. The minimum horizontal distance from the back of the wall facing to the front edge of a drilled shaft should be 3 ft (1 m) in order to have enough room for proper back-fill compaction.
- 3. For steel-strip reinforced MSE abutments, the minimum horizontal distance from the back of the wall facing to the front edge of the pile should be equal to the diameter of the pile but not less than 3 ft (1 m).
- 4. In situations where the pile is anticipated to interfere with the reinforcement, specific methods for pile installation must be developed to overcome this problem. Simple cutting and bending of reinforcement to facilitate pile installation should not be allowed.

2.5 Tolerable Vertical and Horizontal Displacements

2.5.1 Criteria for Tolerable Settlements and Angular Distortions

The settlement of bridge foundations is typically expressed in terms of the angular distortion, which is defined

TABLE 2.1 Allowable angular distortion of bridge foundations

as the differential settlement divided by the span length. Uneven settlements of bridge abutments and piers can affect ride quality, functioning of deck drainage, structural integrity, and aesthetics of the bridge. Such movements often lead to costly maintenance and repair measures. Table 2.1 summarizes the allowable angular distortions of simple span and continuous span bridges from various studies and reports.

According to the bridge design guidelines of the Arizona Department of Transportation (AZDOT, 2009), the total settlement of a bridge foundation per 30 m (100 ft) span should be limited to 13 mm (0.5 in). AZDOT (2009) also states that: (1) higher total settlement limits may be used if the superstructure is adequately designed for such settlements, (2) factors such as rideability and aesthetics should be checked by the designer, and (3) total settlement greater than 2.5 in (63.5 mm) per 100 ft (30 m) span must be approved by the state bridge division.

Based on tolerable movement analyses of 148 highway bridges supported by spread footings on compacted backfill throughout Washington, Dimillio (1982) found that no serious distress was observed for bridges that experienced 1 to 3 in. (25.4 to 76.2 mm) of

		Reference	
Type of bridge	Moulton et al. (1982) and Elias et al. (2001)	Moulton et al. (1985)	AASHTO (2012)
Simple span Continuous span	0.005 0.004	0.007 0.004	0.008 0.004



Figure 2.2 Tolerable movements for bridge foundations (Bozozuk, 1978; adapted from Salgado, 2008).

differential settlement. Based on field studies of 314 bridges and theoretical analyses, Moulton, Ganga Rao, and Halvorsen (1982) concluded that the bridges that performed acceptably experienced an average settlement of 2 in. (50.8 mm).

2.5.2 Criteria for Tolerable Horizontal Displacements

Horizontal displacements cause more severe and widespread problems for highway bridge structures than do equal magnitudes of vertical movement. Figure 2.2 shows a compilation made by Bozozuk (1978) of acceptable and unacceptable horizontal and vertical bridge foundation movements. It can be observed that the

TABLE 2.2

MSE	abutments:	case	histories	and	field	load	tests	
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tolerable vertical displacement is as much as 100 mm (3.9 in.), but the tolerable horizontal displacement is only 50 mm (2 in.) in order to avoid major service-ability problems.

According to Wahls (1983), horizontal displacements in excess of 2 in. are likely to cause structural distress. Moulton, Ganga Rao, and Halvorsen (1985) observed that horizontal displacements cause more damage when accompanied by vertical settlements than when occurring alone. According to Moulton et al. (1985), horizontal displacements of less than 1 in. (25 mm) were tolerable, while those greater than 2 in. were considered to be intolerable. As a result, Moulton et al. (1985) suggested that the horizontal displacement be limited to 1.5 in. (38 mm).

Reference	Location	Abutment type and height	Facing details	Backfill type and properties	Reinforcement type and properties	Footing/pile details
Miyata and Kawasaki (1994)	Japan	Direct $(H = 5 \text{ m})$	Gabions	Not reported	FRP geogrid ($L = 4.5 \text{ m},$ $s_v = 0.25-2 \text{ m},$ $T_u = 49 \text{ kN/m}$)	Footing ($B = 2.3 \text{ m}, D = 0,$ $x = 1 \text{ m}, q_b = 127 \text{ kPa}$)
Benigni et al. (1996)	Trento, Italy	Direct $(H = 5 \text{ m})$	Geotextile wrapping	Sandy gravelly soil $(\gamma_d = 19.6-20.4 \text{ kN/m}^3, wc = 2.4-5.5\%$ $c_{tx} = 100 \text{ kPa}, \phi_{tx} = 40^\circ)$	Polyfelt PEC 50/25 ($L = 2 \text{ m}, s_v = 0.5 \text{ m}, T_u = 27 \text{ kN/m}$)	Footing $(B = 3 \text{ m}, L_f = 3 \text{ m}, D = 0,$ $x = 0.6 \text{ m}, q_b = 84 \text{ kPa})$
Gotteland et al. (1997)	France	Direct $(H = 4.4 \text{ m})$	Segmental concrete blocks	Fine sand ($\gamma_d = 16.6 \text{ kN/m}^3$, RC = 100%, $c = 2 \text{ kPa}, \phi = 30^\circ$)	Non-woven geotextile (L = 2.5 m, $s_v = 0.29 \text{ m},$ $T_u = 25 \text{ kN/m})$	Footing (B = 1 m, D = 0, x = 1.5 m)
Ketchart and Wu (1997)	Denver, Colorado, USA	Direct $(H = 7.6 \text{ m})$	Segmental concrete blocks	Road base material ($\gamma_d = 19.1 \text{ kN/m}^3$, RC = 90%, $wc = 1.6%$)	Woven PP geotextile ($s_v = 0.2 \text{ m},$ $T_u = 38 \text{ kN/m}$)	Footing $(B = 2.4 \text{ m}, L_f = 3.7 \text{ m}, t = 0.3 \text{ m}, D = 0, x = 0.2 \text{ m}, q_b = 130 \text{ kPa})$
Abu-Hejleh et al. (2000)	Denver, Colorado, USA	Direct $(H = 6.4 \text{ m})$	Mesa blocks (457 \times 279 \times 203 mm, $f_c = 28$ MPa)	CDOT class-1 backfill SW-SM ($\gamma_d = 21 \text{ kN/m}^3$, $RC = 95\%$, $wc = 5.6\%$, $c_{tx} = 70 \text{ kPa}$, $\phi_{tx} = 40^\circ$)	UX-PE Geogrid ($L = 7.8-13 \text{ m}$, $s_v = 0.4 \text{ m}$, $T_u = 157.3 \text{ kN/m}$)	Footing $(B = 3.81 \text{ m}, t = 0.61 \text{ m}, x = 1.35 \text{ m}, q_b = 150 \text{ kPa})$
Keller and Devin (2003)	Plumas National Forest, California, USA	Direct (H = 1.5 and 2.4 m)	Timber (0.15 m× 0.15 m)	Onsite rocky soil $(RC = 95\%)$	Woven polyester geotextile $(L = 2 \text{ m}, s_v = 0.15 \text{ m}, T_u = 52-70 \text{ kN/m})$	Footing $(B = 0.3 \text{ m}, L_f = 0.3 \text{ m}, D = 0.15 \text{ m}, x = 0.3 \text{ m})$
Berg and Vulova (2007)	Expressway- 470, Colorado, USA	Mixed $(H = 4.6 \text{ m})$	Full-height precast concrete panels	Silty sand (properties not reported)	UX HDPE geogrid ($s_v = 0.9$ m, FS for connection strength = 1.3–7.7)	H-pile (HP12 × 74, $x = 0.8$ –1.4 m)
Helwany et al. (2007)	McLean, Virginia, USA	Direct $(H = 4.7 \text{ m})$	Cinder blocks (397 × 194 × 194 mm)	Non-plastic silty sand SP-SM ($RC = 99\%$, $wc = 13\%$, $c_{tx} = 20$ kPa, $\phi_{tx} = 37^{\circ}$)	Woven PP geotextile (L = 3.15 m, $s_v = 0.2 \text{ m},$ $T_u = 21-70 \text{ kN/m})$	Footing $(B = 0.9 \text{ m}, L_f = 4.5 \text{ m}, t = 0.3 \text{ m}, D = 0, x = 0.15 \text{ m}, q_b = 200-800 \text{ kPa})$
Nelson (2013)	Provo, Utah, USA	Mixed $(H = 6.8 \text{ m})$	WWM panels + geofabric (3 m × 1.5 m)	Sandy gravel A-1-a $(\gamma_d = 14.1 \text{ kN/m}^3,$ RC = 97.4%, wc = 4.8%)	Ribbed steel strips (50 mm \times 3 mm, L = 8.5 m, $s_v = 0.6$ m)	CEP pile (OD = 324 mm, $t_w = 9.5$ mm, s = 2.1 m, $x = 0.24$ –1.9 m)

2.5.3 Tolerable Deformation Criteria for Reinforced Bridge Support

MSE walls can tolerate larger total and differential settlements than rigid walls. The amount of total and differential settlements that can be tolerated depends on the wall facing material, configuration, and timing of facing construction (AASHTO, 2012). AASHTO (2012) states that abutments should not be constructed on MSE walls if the anticipated angular distortion is greater than 50 percent of the values recommended by Moulton et al. (1985), as shown in Table 2.1. Based on the results of performance tests or mini-pier experiments of geosynthetic-reinforced soil (GRS) abutments, FHWA (2012) states that the vertical strain of the GRS mass should be limited to 0.5%, while the horizontal strain should be limited to 1%.

2.6 MSE Abutments: Case Histories and Field Load Tests

Table 2.2 lists some case histories and field load tests pertaining to MSE abutments. The table provides information about the abutment location and type (direct or mixed), facing height H and type (gabions, modular blocks, geotextile wrapping, precast panels, and timber), backfill type and properties (compacted dry unit weight γ_d , relative compaction RC, placement water content wc, cohesive intercept c_{tx} and peak friction angle ϕ_{tx} from triaxial tests performed on samples compacted to the same *RC* value as that of the backfill), type of reinforcement [woven/non-woven polypropylene (PP) geotextiles, uniaxial (UX), high-density polyethylene (HDPE), fiberglass reinforced plastic (FRP) geogrids, ribbed steel strips and welded wire (WW) grids] and reinforcement properties (length L, vertical spacing s_v , horizontal spacing s_h and tensile strength T_{μ} from wide-width tensile tests), footing details (width B, length L_t , thickness t, depth of embedment D, horizontal distance x from the back of the wall facing to the front edge of the footing, and footing pressure q_b), pile type [closed-ended pipe (CEP) pile, open-ended pipe (OEP) pile, and H-pile) and pile properties [outer diameter (OD), wall thickness t_w , horizontal distance x from the back of the wall facing to the front edge of the pile, and center-to-center spacing *s* between piles).

3. FINITE ELEMENT ANALYSIS

3.1 Problem Definition

A direct MSE abutment of height *H* retains backfill soil that is reinforced with ribbed steel strips of length *L* with vertical spacing s_v and horizontal spacing s_h , as shown in Figure 3.1. A footing of width *B* is embedded at a depth *D* within the backfill at a distance *x* behind the back of the wall facing. In this study, the height of the abutment is 9 m and the precast concrete facing panels have dimensions of 3 m × 1.5 m and thickness of 150 mm. The cross-section of the reinforcement is 50 mm × 4 mm. The width and thickness of the levelling pad are 0.3 m and 150 mm, respectively, and those of the footing are 2 m and 150 mm, respectively.

3.2 Backfill Constitutive Model

The advanced two-surface bounding plasticity constitutive model developed by Loukidis and Salgado (2009) was used to model the mechanical response of the backfill material. A user-defined model subroutine VUMAT, which can be implemented in ABAOUS/ Explicit (ABAQUS, 2012), was coded in FORTRAN to describe the constitutive relationships. Table 3.1 provides the model parameters for clean, dry-deposited/airpluviated Toyoura sand and slurry-deposited/water pluviated Ottawa sand. Since the stiffness of the footing, precast facing elements, levelling pad, and steel strip reinforcement are significantly higher than that of the backfill, they have been simulated as linearelastic materials with modulus of elasticity and Poisson's ratio of 25 GPa and 0.2 for the facing panels, levelling pad and footing (Zevgolis & Bourdeau, 2007), and 200 GPa and 0.3 for the reinforcement (Damians et al., 2015).

3.3 Finite Element Mesh

Figure 3.2 shows the configuration of the mesh used for the finite element (FE) analyses performed in Abaqus/CAE 6.12-1 (ABAQUS, 2012). The backfill and facing zones were discretized using 6-noded, bi-quadratic triangular elements, while the reinforcement was modeled using 3-noded, quadratic truss elements.

It is well known that analyses involving materials that soften and follow a non-associative flow rule suffer from the problem of solution non-uniqueness. As the mesh gets refined, the results of the FE analysis change, and convergence to a unique solution does not happen. To overcome this problem, FE analyses should either employ a regularization approach (such as Cosserat or gradient plasticity) or use meshes with element sizes that are consistent with the known shear band thickness (Loukidis & Salgado, 2012). In this study, the mesh around the vicinity of the footing was refined such that the thickness of the elements was $10D_{50}$ of the backfill, which is 2 mm for Toyoura sand and 4 mm for Ottawa sand, in order to capture the formation of shear bands (Abedi, Rechenmacher, & Chupin, 2010; Alshibli & Sture, 1999; Loukidis & Salgado, 2008; Nemat-Nasser & Okada, 2001).

Given the substantial roughness of the interface between the base of the footing and the backfill due to the pouring of concrete, shearing is assumed to happen within the backfill surrounding the footing. Therefore, perfect contact, which means the common nodes of the footing and the backfill are tied to each other with respect to all degrees of freedom, was assumed at the footing-backfill interface. A similar approach was used for the reinforcement-backfill interface assuming the reinforcement elements to be perfectly bonded to the backfill. This is consistent with measured pullout test



Figure 3.1 Schematic illustration of direct MSE abutment.

data for ribbed steel strips and well-compacted granular soils reported in the literature (Balunaini, 2009; Balunaini & Prezzi, 2010; Bathurst, Huang, & Allen, 2011; Miyata & Bathurst, 2012).

The lateral boundaries of the FE mesh were located at distances of 4H behind the wall and H in front of the wall (Zheng & Fox, 2016), and were fixed in the horizontal direction but free to move in the vertical direction via roller supports. The bottom boundary of the FE mesh was fixed in both the horizontal and vertical directions by means of hinges. After equilibrium was achieved between the predefined stress field and the gravity load applied to the whole domain by means of a geostatic step, the footing was loaded in increments until the desired vertical stress was reached. The vertical stress due to the region of soil below the approach slab was simulated by means of a surcharge.

3.4 Modeling of Reinforcement

The reinforcement strip layers in the abutment were modeled as continuous sheet elements. These elements were assigned a constant axial stiffness EA based on the elastic modulus E and cross-sectional area A of the steel strips, and the number N of steel strips per length of the facing (Damians et al., 2015; Zevgolis & Bourdeau, 2007). In order to model them properly, the equivalent properties corresponding to a distinct strip of a sheet were determined and then normalized per linear meter of the facing. The axial stiffness *S* of one unique strip is given by:

$$S = \frac{EA}{L} \tag{3.1}$$

For N distinct strips per linear meter, the combined stiffness S_N of the group of strips is given by:

$$S_{N} = \sum_{i=1}^{N} \frac{E_{i}A_{i}}{L_{i}} = N \frac{EA}{L}$$
(3.2)

Now, the equivalent stiffness of a continuous sheet element S_{eq} is given by:

$$S_{eq} = \frac{E_{eq}A_{eq}}{L_{eq}} \tag{3.3}$$

In order for the axial stiffness of an equivalent sheet that has a width of one linear meter to be equal to the summation of the axial stiffnesses of the individual steel strips that are contained within that one linear meter, the following condition needs to be satisfied.

$$S_N = S_{eq} \Leftrightarrow (EA)_{eq} = N(EA) \tag{3.4}$$

TABLE 3.1 Sand model parameters (Loukidis & Salgado, 2009)

		Parameter		
	Parameter symbol	Toyoura sand	Ottawa sand	Test used for calibration
Small-strain parameters	ν	0.15	0.15*	Tests using local strain transd., or iso comp. or 1-D comp. tests with unloading path
	C_{g}	900	611	Bender element or resonant column tests
	n _g	0.40	0.437	Bender element or resonant column tests
	71	0.001	0.00065	Resonant column tests or triaxial tests with local strain measurements
	α_I	0.40	0.47	Undrained triaxial compression tests
Critical state	Γ_c	0.934	0.78	Triaxial compression tests
	λ	0.019	0.081	Triaxial compression tests
	ξ	0.70	0.20	Triaxial compression tests
	M_{cc}	1.27	1.21	Triaxial compression tests
Bounding surface	k_b	1.5	1.9	Triaxial compression tests
Dilatancy	D_0	0.90	1.31	Triaxial compression tests
	k_d	2.8	2.2	Triaxial compression tests
Plastic modulus	h_I	1.62	2.20	Triaxial compression tests
	h_2	0.254	0.240	Triaxial compression tests
	e_{lim}	1.00	0.81	Triaxial compression tests
	μ	2.0	1.2	Undrained triaxial compression tests
Stress-induced anisotropy	C_I	0.72	0.71	Triaxial extension tests
	<i>C</i> ₂	0.78	0.78	Simple shear or other plane-strain tests
	n_s	0.35	0.35	Simple shear or other plane-strain tests
Inherent anisotropy	α	0.29	0.31	Triaxial compression tests
	k_h	0.11	0.39	Triaxial compression tests
Yield surface	т	0.05	0.05	

*Assumed value.



Figure 3.2 Configuration of finite element mesh.

If s_h is the horizontal spacing of the strips, then N will be equal to $1/s_h$, and thus equation (3.4) becomes:

$$(EA)_{eq} = \frac{1}{s_h}(EA) \tag{3.5}$$

For steel strips with a cross-section of 50 mm × 4 mm, the equivalent thickness t_{eq} of the sheet is equal to $0.2/s_h$ (mm). In the FE analysis, the horizontal spacing of the steel strips s_h and the number N of strips per linear meter of the facing were considered to be equal to 0.5 m and 2, respectively. Therefore, the thickness and stiffness of the equivalent sheet works out to be 0.4 mm (instead of 4 mm, which is the thickness of one strip) and 80×10^3 kN/m, respectively.

4. RESULTS AND DISCUSSION

A parametric study was conducted to investigate the behavior of the direct MSE abutment, including the effects of

- a. normalized horizontal distance from the back of the wall facing to the front edge of the footing (x/H = 0.05, 0.1, 0.15, 0.2, 0.25, 0.3 and 0.5);
- b. relative density of the backfill within a distance of 1 m behind the wall facing ($D_R = 50, 60, 70, 80$ and 90%);
- c. normalized length of reinforcement (L/H = 0.5, 0.7, 0.9 and 1.1);
- d. vertical spacing of reinforcement ($s_v = 0.2, 0.4$ and 0.6 m); and
- e. bridge load ($q_b = 100, 200, 300, 400$ and 500 kPa).

The backfill was considered to be Ottawa sand (SP), a silica sand with grain sizes ranging from 0.1 to 0.6 mm, mean particle size D_{50} of 0.39 mm, uniformity

coefficient C_u of 1.43, specific gravity G_s of 2.65, minimum void ratio e_{min} of 0.48, maximum void ratio e_{max} of 0.78, and critical-state friction angle ϕ_c of 30.2° (Murthy, Loukidis, Carraro, Prezzi, & Salgado, 2007). Results are expressed in terms of horizontal displacement profiles of the facing, reinforcement strain profiles and load-settlement response curves of the footing.

4.1 Effect of Normalized Horizontal Distance of Footing Behind Wall Facing

Figure 4.1 shows the effect of the normalized horizontal distance x/H from the back of the wall facing to the front edge of the footing on the profiles of normalized horizontal displacement δ_h/H of the wall facing, for B = 2 m, D/B = 0.25, L/H = 0.7, $s_v = 0.4$ m, $s_h = 0.5 \text{ m}, D_R = 90\%$, and $q_b = 200 \text{ kPa}$. It is observed that most of the horizontal displacements of the wall facing are concentrated within the upper half of the abutment. When the footing is located relatively close to the facing (x/H = 0.05), the maximum normalized horizontal displacement of the facing is 0.3%, and occurs at a normalized facing elevation z/H of 0.9, which corresponds to the location where the footing is embedded. As x/H increases, the footing moves farther and farther away behind the wall facing and hence δ_h/H decreases. For instance, at the top of the facing (z/H = 1), δ_h/H decreases by a relatively small amount (2.5%) as x/H increases from 0.05 to 0.1, but decreases by as much as 23% as x/H increases from 0.1 to 0.2. It is interesting to note that as x/H increases, the facing elevation corresponding to the maximum horizontal displacement, shifts from z/H of 0.9 to somewhere closer to the mid-height of the facing (z/H = 0.5). This



Figure 4.1 Normalized horizontal displacement profiles of wall facing – effect of x/H.

is because the influence of the footing on the horizontal displacement of the wall facing decreases as the footing moves farther behind the facing. However, it should be noted that keeping the footing far away from the wall facing in order to benefit from the reduced horizontal displacement of the facing is not practically feasible since it increases the overall span of the bridge. Hence, the footing should be located at an optimum distance behind the wall facing in order to minimize the horizontal displacement of the facing as much as possible, without significantly increasing the length of the bridge.

Figure 4.2 shows the variation of the normalized settlement w/B of the footing with the normalized distance x/H of the footing behind the wall facing for B = 2 m, D/B = 0.25, L/H = 0.7, $s_v = 0.4$ m, $s_h = 0.5$ m, $D_R = 90\%$, and $q_b = 200$ kPa. Figure 4.2 indicates that the optimum distance of the footing behind the wall facing is between 0.15 and 0.2 times the height of the wall facing, since the settlement of the footing follows a decreasing path. Further, it is observed that increasing the horizontal distance of the footing from 0.2 to 0.25 times the height of the wall facing does not reduce the settlement of the footing by as much as that when the distance is increased from 0.15 to 0.2 times the height of the wall facing.

4.2 Effect of Relative Density of Backfill

The relative density of the backfill within 1 m behind the wall facing was varied from 50% to 90% while keeping the relative density of the backfill at other locations equal to 90%. This was done in order to capture the use of lighter compaction equipment, such as plate compactors instead of conventional rollers, behind the wall facing (see Figure 4.3). A similar approach was adopted by Damians, Bathurst, Josa, Lloret, and Albuquerque (2013) and Bathurst et al. (2015) to simulate the construction of steel strip-reinforced MSE walls using the finite element method.

Figure 4.4 shows the effect of the relative density D_R of the backfill within 1 m behind the wall facing on the profiles of normalized horizontal displacement δ_h/H of the wall facing for B = 2 m, D/B = 0.25, x/H = 0.2, L/H = 0.7, $s_v = 0.4$ m, $s_h = 0.5$ m, and $q_b = 200$ kPa. As expected, the normalized horizontal-facing displacements decrease with increasing relative density of the backfill. The maximum reduction was found to be 22% at a normalized wall elevation z/H of 0.8 when D_R was increased from 50% to 90%.

Figure 4.5 shows the effect of the relative density D_R of the backfill within 1 m behind the wall facing on the load-settlement response of the footing for B = 2 m, D/B = 0.25, x/H = 0.2, L/H = 0.7, $s_v = 0.4$ m, and $s_h = 0.5$ m. For a footing pressure of 200 kPa, the normalized settlement w/B of the footing decreases by 17.5% as D_R increases from 50% to 90%.

4.3 Effect of Normalized Length of Reinforcement

Figure 4.6 shows the effect of the normalized length L/H of the reinforcement on the profiles of normalized horizontal displacement δ_h/H of the wall facing, for B = 2 m, D/B = 0.25, x/H = 0.2, $s_v = 0.4 \text{ m}$, $s_h = 0.5 \text{ m}$, $D_R = 90\%$, and $q_b = 200 \text{ kPa}$. At a normalized wall elevation z/H of 0.5, it is observed that the normalized horizontal displacement of the wall facing decreases by 28%, 15%, and 12% as L/H increases from 0.5 to 0.7, 0.7 to 0.9, and 0.9 to 1.1, respectively. Thus,



Figure 4.2 Optimum distance of footing behind wall facing.



Figure 4.3 Compaction of backfill immediately behind the facing using plate compactor.



Figure 4.4 Normalized horizontal displacement profiles of wall facing – effect of D_R .

Figure 4.6 indicates that, for the conditions investigated, the industry default value of 0.7H for the reinforcement length is a good starting point for preliminary design of direct MSE abutments under static loading. The length of the reinforcement may be increased beyond 0.7H depending on the magnitude of the bridge load and the height of the abutment.

4.4 Effect of Vertical Spacing of Reinforcement

Figure 4.7 shows the effect of the vertical spacing s_v of the reinforcement on the profiles of normalized horizontal displacement δ_h/H of the wall facing, for $B = 2 \text{ m}, D/B = 0.25, x/H = 0.2, L/H = 0.7, s_h = 0.5 \text{ m}, D_R = 90\%$, and $q_b = 200 \text{ kPa}$. The closer the spacing of



Figure 4.5 Load-settlement response of footing – effect of D_{R} .



Figure 4.6 Normalized horizontal displacement profiles of wall facing – effect of L/H.

the reinforcement, the smaller the horizontal displacement of the facing is. At a normalized wall elevation z/H of 0.8, the normalized horizontal displacement of the facing decreases by as much as 20% as s_{ν} decreases from 0.6 to 0.2 m.

Figure 4.8 shows the effect of the vertical spacing s_v of the reinforcement on the load-settlement response of

the footing for B = 2 m, D/B = 0.25, x/H = 0.2, L/H = 0.7, $s_h = 0.5$ m, and $D_R = 90\%$. For a footing pressure of 200 kPa, it is observed that the normalized settlement w/B of the footing decreases by 20.5% as s_v decreases from 0.6 to 0.2 m.

Figure 4.9 shows the distribution of axial strains in the reinforcement for vertical spacings of 0.2, 0.4



Figure 4.7 Normalized horizontal displacement profiles of wall facing – effect of s_{ν} .



Figure 4.8 Load-settlement response of footing – effect of s_{v} .

and 0.6 m with B = 2 m, D/B = 0.25, x/H = 0.2, L/H = 0.7, $s_h = 0.5$ m, and $D_R = 90\%$. The specific reinforcement layer under consideration is located at the mid-height of the wall facing. It should be noted that x/H is the normalized horizontal distance from the back of the wall facing to the front edge of the footing,

whereas X/H is the normalized horizontal distance measured from the back of the wall facing to any point located within the backfill. Referring to Figure 4.9, it is observed that for a given vertical spacing of the reinforcement, the axial strain in the reinforcement is maximum at the reinforcement-panel connection (X/H = 0)



Figure 4.9 Reinforcement strain profiles – effect of s_{ν} .



Figure 4.10 Normalized horizontal displacement profiles of wall facing – effect of q_b .

and decreases as we go farther away behind the wall facing. Moreover, the axial strain in the reinforcement decreases with the vertical spacing of the reinforcement. For instance, at the reinforcement-panel connection (X/H = 0), the axial strain in the reinforcement decreases by 19% as s_{y} decreases from 0.6 to 0.2 m.

4.5 Effect of Bridge Load

Figure 4.10 shows the effect of the magnitude of bridge load q_b on the profiles of normalized horizontal displacement δ_h/H of the wall facing, for B = 2 m, D/B = 0.25, x/H = 0.2, L/H = 0.7, $s_v = 0.4$ m,



Figure 4.11 Reinforcement strain profiles – effect of q_b .

 $s_h = 0.5$ m, and $D_R = 90\%$. It is observed that the magnitude and location of the maximum horizontal displacement of the wall facing, and the deflected shape of the facing, depend on the magnitude of the bridge load. For relatively small magnitudes of bridge load ($q_b = 100$ kPa), the maximum normalized horizontal displacement of the wall facing is 0.22% at a normalized wall elevation z/H of about 0.6. The corresponding deflected shape of the facing resembles somewhat like a concave/convex shape due to the relatively small magnitude of the load. On the other hand, for relatively large magnitudes of bridge load ($q_b = 500$ kPa), the maximum normalized horizontal displacement of the wall facing is 0.46% (which is 2.1 times that of the value for q_b equal to 100 kPa) and occurs at the top of the wall (z/H = 1). The wall facing deflects away from the backfill due to the lateral displacement of the soil below the footing.

Figure 4.11 shows the effect of the magnitude of bridge load q_b on the distribution of axial strains in the reinforcement layer located at mid-height of the wall facing, for B = 2 m, D/B = 0.25, x/H = 0.2, L/H = 0.7, $s_v = 0.4$ m, $s_h = 0.5$ m, and $D_R = 90\%$. The axial strain in the reinforcement increases by 27.6% at the reinforcement-panel connection (X/H = 0) and by 20.4% at the free end of the reinforcement (X/H = 0.7) as q_b increases from 100 to 500 kPa.

5. VALIDATION WITH EXPERIMENTAL RESULTS

5.1 Laboratory Model Test (Hirakawa et al., 2004)

Hirakawa, Takaoka, Tatsuoka, and Uchimura (2004) performed a series of laboratory model tests on a

45-cm-high, geogrid-reinforced soil retaining wall loaded on the crest over a 10-cm-wide rigid strip footing (Figure 5.1 and Figure 5.2). Air-dried Toyoura sand with specific gravity G_s of 2.65, dry unit weight γ_d of 16 kN/m³, maximum void ratio e_{max} of 0.933, and minimum void ratio e_{min} of 0.624, was compacted at a relative density D_R of 90% to serve as the backfill. The backfill was reinforced with 8 layers of 40-cm-long polyester geogrids placed at a vertical spacing of 5 cm (Figure 5.2).

Figure 5.3 depicts the shape and structure of the geogrid reinforcement that was used in the tests. The geogrid consists of 1-mm-wide longitudinal and transverse members with an aperture size of 1.8×1.8 cm. The ultimate tensile strength of the geogrid measured at a strain rate of 1%/min is 39.2 kN/m at a failure strain less than 22% in both the longitudinal and transverse directions. The footing was located at a setback of 10 cm from the back of the facing. Figure 5.4 compares the predicted reinforcement tension profiles obtained from the FE analysis with those measured by Hirakawa et al. (2004) for the 5th geogrid layer measured from the top of the wall. Comparisons are made for footing pressures of 50, 100 and 200 kPa. It is observed that the simulation results compare reasonably well with the test data.

5.2 Full-Scale Test (Wu, 1992)

Wu (1992) presented results from a full-scale test of a 3-m-high geosynthetic-reinforced soil wall in Denver. Figure 5.5 shows the configuration of the Denver wall. The wall facing was comprised of inter-connected timber logs with plywood packing.



Figure 5.1 Test setup of reinforced soil retaining wall (Hirakawa et al., 2004).



Figure 5.2 Cross-section of reinforced soil wall model (Hirakawa et al., 2004).



Figure 5.3 Shape and structure of geogrid reinforcement (Hirakawa et al., 2004).

The elastic modulus and shear modulus of the beam section of the timber facing were 36 MPa and 13.9 MPa, respectively. Air-dried Ottawa sand with specific gravity of 2.65, maximum and minimum unit weights of 17.7 kN/m³ and 15.3 kN/m³, respectively, was used as the backfill and placed using the air pluviation method at a unit weight of 16.8 kN/m³. Lightweight, nonwoven, heat-bonded, polypropylene geotextile with modulus at 10% elongation of 4.45 kN, and 60% elongation at break, was used as the reinforcement. The length and vertical spacing of the reinforcement were 1.67 m and 0.28 m, respectively. After construction of the wall, a surcharge was applied to the top surface of the backfill using a pair of air bags. Figure 5.6 compares the predicted horizontal displacement profiles of the wall facing with those measured by Wu (1992) for surcharge pressures of 100 and 200 kPa. Predictions are in good agreement with the measured response of the wall and thus validate the FE results obtained.



Figure 5.4 Comparison with measured reinforcement tension profiles of Hirakawa et al. (2004).



Figure 5.5 Configuration of Denver wall (Wu, 1992).



Figure 5.6 Comparison with measured wall displacement profiles of Wu (1992).

6. STATE-OF-PRACTICE SURVEY

6.1 Introduction

An agency survey was carried out to obtain information on the current state-of-practice of MSE abutments in various States in the U.S. An email solicitation was distributed to all the 50 Departments of Transportation (DOTs). The following information was requested:

- 1. The percentage of construction, and height, of direct and mixed MSE abutments.
- 2. The type and dimensions of the facing and the reinforcement used in direct and mixed MSE abutments.
- 3. The depth of embedment of the footing, the horizontal distance from the back of the facing to the front edge of the footing, and the vertical clearance between the bottom of the footing and the top level of the reinforcement in direct MSE abutments.
- 4. The type, dimensions, and center-to-center spacing of piles and the horizontal distance from the back of the facing to the front edge of the piles in mixed MSE abutments.
- 5. The problems observed with regard to construction and performance of MSE abutments.

Replies were received from 31 DOTs, which provided answers for the above points, thus providing complete data. This chapter presents and discusses the information provided by these DOTs.

6.2 Results of Survey

Thirty-one DOTs responded to the survey questionnaire: Arizona, California, Colorado, Connecticut, Delaware, Florida, Georgia, Idaho, Indiana, Illinois, Louisiana, Maine, Maryland, Massachusetts, Michigan, Missouri, Montana, Nebraska, Nevada, New Jersey, New York, North Carolina, North Dakota, Ohio, Oklahoma, Oregon, South Carolina, South Dakota, Virginia, West Virginia and Wisconsin. 83.9% of the DOTs reported to have constructed MSE abutments in their respective States while 16.1% reported on the contrary. The respondent from California reported that only two sets of steel strip-reinforced MSE abutments (one direct and one mixed) are currently in service as part of demonstration projects. The mixed MSE abutment has a vertical face with 5 ft square facing panels while the direct MSE abutment has a stepped block facing with planting on the steps in a landscaped urban interchange.

The respondent from Connecticut reported to have not used traditional MSE abutments, but is currently piloting geosynthetic-reinforced soil integrated bridge system (GRS-IBS) abutments designed according to FHWA (2012). The respondents from North Dakota and Oklahoma mentioned no construction of MSE abutments in their States. However, Oklahoma DOT mentioned that they are considering the use of MSE abutments to supplement their existing methodology of using pile-supported abutments with cast-in-place walls for soil retention. Table 6.1 shows the percentage of use of direct and mixed MSE abutments given by the other respondents. It is observed that mixed MSE abutments have been constructed more often than direct MSE abutments in the United States.

Figure 6.1 shows the range of heights of direct and mixed MSE abutments and their relative percentages.

TABLE 6.1						
Use of direct	and mixed	MSE	abutments	by	various	DOTs

DOT	Direct abutment (%)	Mixed abutment (%)	Other (%)
Arizona	0	100	
Colorado	<10	< 60	38 (deep foundation with slope pavement)
Florida	5	95	_
Georgia	1	99	
Idaho	10	90	
Indiana	0	100	
Illinois	3	97	
Louisiana	5	95	_
Maine	45	55	-
Maryland	0	100	-
Massachusetts	<1	<1	4 GRS-IBS bridges
Michigan	1	5	6
Missouri	1	99	-
Montana	0	100	-
Nebraska	5	95	-
Nevada	0	100	-
New Jersey	1	5	_
North Carolina	1	99	-
Ohio	0	100	-
Oregon	2	98	-
South Carolina	0	100	-
South Dakota	5	95	_
Virginia	1	99	-
West Virginia	1	99	_
Wisconsin	2	98	



Figure 6.1 Range of heights of direct and mixed MSE abutments.

It is observed that the heights of most MSE abutments in the United States fall within the range of 21–30 ft. In fact, 63.9% and 69.2% of the DOTs have constructed direct and mixed MSE abutments, respectively, with heights of 21–30 ft., while 30.6% and 21.2% of the DOTs have constructed direct and mixed MSE abutments, respectively, with heights of 10–20 ft. A few MSE abutments have been constructed with heights of 31–40 ft. (5.6% and 7.7% of the DOTs for direct and mixed MSE abutments, respectively). It can also be observed that none of the DOTs have constructed direct MSE abutments in the 41–50 ft. height category, however, 1.9% of the DOTs have constructed mixed MSE abutments with heights of 41–50 ft.

Figure 6.2 and Figure 6.3 show the percentages of different types of facing used in direct and mixed MSE



Figure 6.2 Percentages of different types of facing used in direct MSE abutments.



Figure 6.3 Percentages of different types of facing used in mixed MSE abutments.

abutments, respectively. Precast panels are indicated in blue color, modular blocks in brown color, geotextile in green color, and other types of facing (such as sheet pile, gabion, welded wire face) in yellow color. It is observed that nine out of eighteen DOTs use only precast panel facing for direct MSE abutments, while the rest of the DOTs use a combination of the aforesaid types of facing. For instance, 50% of the direct MSE abutments in Colorado consist of precast panels as the facing element, 40% with modular blocks, 8% with geotextile wrap-around and 2% with sheet piles. In the case of mixed MSE abutments, 13 out of 26 DOTs use only precast panels for the facing while the rest use mostly precast panels with small percentages of the other types of facing listed. The typical dimensions (length \times width \times thickness) of precast panels used by the DOTs are $10' \times 5' \times 6''$ and $5' \times 5' \times 6''$, while those of modular blocks are $46'' \times 28'' \times 18''$, $22'' \times 12'' \times 8''$, $16'' \times 10'' \times 8''$ and $16'' \times 8'' \times 8''$.

Figure 6.4 and Figure 6.5 show the percentages of different types of reinforcement used in direct and mixed MSE abutments, respectively. Steel strips are indicated in blue color, wire mesh in brown color, woven geotextile in green color, and uniaxial geogrid in yellow color. It can be seen that steel strips are the preferred choice of reinforcement by most DOTs for both direct and mixed MSE abutments. The typical cross-section of the steel strips used by the DOTs is 50 mm \times 4 mm thick, while the length of the strips ranges from 0.7–0.8 times the height of the abutment.

Figure 6.6 shows the ranges of the clear horizontal distance from the back of the facing to the front edge of the footing in a direct MSE abutment (denoted by x in

Figure 3.1). 46.4% of the DOTs reported the distance to be within 2 ft., while 39.3% reported it to be within 2.1–4.0 ft. A small percentage of the DOTs (14.3%) reported having used a relatively higher distance of 4.1–6.0 ft. between the back of the facing and the front edge of the footing. The minimum requirement specified by FHWA (2009) is 0.5 ft.

Figure 6.7 shows the ranges of the depth of embedment of the footing in a direct MSE abutment (denoted by *D* in Figure 3.1). 37.5% of the DOTs reported the depth of embedment of the footing to be between 1.1 and 2.0 ft., followed by 29.2% for 2.1–3.0 ft., 16.7% for 3.1–4.0 ft., and 8.3% for both the 0.0–1.0 ft. and greater than 4.0 ft. categories. It should be noted that FHWA (2009) does not specify any guidelines for the depth of embedment of the footing.

Figure 6.8 shows the ranges of the vertical clearance between the base of the footing and the topmost reinforcement layer in a direct MSE abutment. 46.2% of the DOTs reported the vertical clearance to be between 0.6 and 1.0 ft., followed by 26.9% for 0.0–0.5 ft., 19.2% for 1.1–1.5 ft., and 7.7% for the 1.6–2.0 ft. category. The minimum vertical clearance between the bottom of the footing and the top level of reinforcement is 1 ft. as per FHWA (2009).

Figure 6.9 shows that 50% of the piles used by the respondents in mixed MSE abutments are partialdisplacement piles (PDP), 31.25% are displacement piles (DP) and 18.75% are non-displacement piles (NDP). The partial-displacement piles consist of H-piles (78.1%) and open-ended pipe piles (21.9%), the displacement piles consist of closed-ended pipe piles (80%) and prestressed concrete piles (20%), and the non-displacement piles



Figure 6.4 Percentages of different types of reinforcement used in direct MSE abutments.



Figure 6.5 Percentages of different types of reinforcement used in mixed MSE abutments.



Figure 6.6 Horizontal distance between facing and footing in direct MSE abutments.

comprise drilled shafts. The typical cross-sections of H-piles used by the DOTs are 10×42 , 12×53 , 12×73 , 14×73 , 12×89 , and 14×89 (in. × lb/ft.). The outer diameter of open- and closed-ended pipe piles vary from 10" to 20". The diameter of drilled shafts ranges from 24" to 60". The spacing of the piles varies from 3' to 10' for H-piles and 2 to 3 times the pile diameter for pipe piles and drilled shafts.

Figure 6.10(a) and (b) show the ranges of the clear horizontal distance from the back of the facing to the front edge of the pile in a mixed MSE abutment for driven piles and drilled shafts, respectively.



Figure 6.7 Depth of embedment of footing in direct MSE abutments.



Figure 6.8 Vertical clearance between base of footing and topmost reinforcement layer in direct MSE abutments.

Referring to Figure 6.10(a) for driven piles, it is observed that 53.8% of the DOTs reported the distance to be between 2.1 and 4.0 ft., followed by 28.2% for 4.1–6.0 ft., 7.7\% for 0.0–2.0 ft., and

5.1% each for 6.1-8.0 ft. and greater than 8.0 ft. categories. According to FHWA (2009), the minimum clear horizontal distance between the back of the facing and the front edge of a driven pile is 1.5 ft.



Figure 6.9 Percentages of different types of piles used in mixed MSE abutments based on method of pile installation.

Similarly, referring to Figure 6.10(b) for drilled shafts, it is observed that 61.5% of the DOTs reported the distance to be between 2.1 and 4.0 ft., followed by 23.1% for 4.1–6.0 ft. and 15.4% for the 6.1–8.0 ft. category. According to FHWA (2009), the minimum clear horizontal distance between the back of the facing and the front edge of a drilled shaft is 3 ft.

Figure 6.11 shows the problems experienced by DOTs with respect to MSE abutments in the field. The various problems reported by the DOTs expressed as a percentage in descending order are

- a. loss of backfill material through the joints of the facing (18.1%);
- b. inadequate drainage system (12.5%);
- c. unsatisfactory workmanship and QA/QC (12.5%);
- d. growth of vegetation in the joints of the facing (11.1%);
- e. inconsistent backfill compaction (11.1%);
- f. excessive settlement at bridge approach joint (11.1%);
- g. cracking/deterioration of facing (6.9%);
- exposure of levelling pad due to scour or erosion of soil cover (6.9%);
- non-availability of good quality, free-draining backfill soil locally (5.6%);
- j. corrosion of metallic reinforcement (2.8%); and
- k. improper design of abutment components (1.4%).



(a)



(b)

Figure 6.10 Horizontal distance from back of facing to front edge of piles in mixed MSE abutments: (a) driven pile and (b) drilled shaft.



Figure 6.11 Problems experienced by DOTs with respect to MSE abutments.

7. CONCLUSIONS

7.1 Summary of Work

Reinforced soil structures are cost-effective alternatives for most applications where reinforced concrete or gravity type walls have traditionally been used to retain soil. These include bridge abutments and wing walls, as well as areas where the right-of-way is restricted, such that an embankment or excavation with stable side slopes cannot be constructed. MSE walls have been used extensively for highway infrastructure and provide several advantages over traditional retaining walls, including lower cost, rapid construction, and good performance under static and seismic loading. In addition, MSE walls have been developed as bridge abutments with loads applied either directly to the top of the reinforced soil mass through a shallow footing (direct MSE abutment) or onto piles that are either driven through the reinforced backfill or preinstalled before construction of the MSE wall (mixed MSE abutment).

Although the response of conventional MSE walls has been investigated, both experimentally as well as numerically, by various researchers, comparatively less work has been conducted to study the behavior of MSE walls used as abutments. Numerical analyses (finite element and finite difference analyses) of the behavior of MSE abutments have so far employed relatively simple constitutive models for the backfill such as, Mohr-Coulomb with non-associated flow rule and Duncan-Chang hyperbolic relationship, or cap plasticity model with Drucker-Prager failure criterion. In this study, an advanced two-surface bounding plasticity model based on critical state soil mechanics has been used to simulate the mechanical response of the backfill.

Finite element analyses were performed for a 9-mhigh, steel-strip reinforced direct MSE abutment, using clean Ottawa sand as the backfill material. Ottawa sand was modeled using the Loukidis-Salgado constitutive model and the steel strip reinforcement was modeled as a sheet with equivalent dimensions and stiffness using linear-elastic truss elements. The numerical simulations closely followed the field construction sequence of an MSE abutment. This was done by activating the elements of the facing, backfill, reinforcement, and footing in stages. Simulation results were first validated with laboratory model and full-scale test results of direct MSE abutments available in the literature. The predicted horizontal displacement profiles of the wall facing and reinforcement tension profiles were found to be in good agreement with the measured data.

Results from the parametric study indicate that the horizontal distance from the back of the facing to the front edge of the footing, backfill compaction, reinforcement length and spacing, and bridge load, have significant influence on the horizontal-facing displacements, bridge footing settlements, and axial strains in the reinforcements. For a given bridge load, abutment movements can be reduced by properly compacting backfill soil (especially within the 1 m distance behind the wall facing), decreasing reinforcement spacing, and increasing reinforcement length. Based on the results obtained from the finite element simulations performed in this study, it is recommended that the clear horizontal distance from the back of the wall facing to the front edge of the footing be within 0.15 to 0.2 times the height H of the wall facing measured from the ground surface to the top of the facing, with a minimum of 0.1H. The depth of embedment of the footing is suggested to be within 0.2 to 0.25 times the width of the footing. The minimum vertical clearance between the base of the footing and the top level of reinforcement should be 0.3 m. A reinforcement length of 0.7H is suggested as a reasonable starting point for preliminary design and internal stability analysis of a direct MSE abutment.

A DOT survey was carried out to obtain information on the current state-of-practice of MSE abutments in various States in USA. An email solicitation was distributed to all 50 DOTs, however, responses were received from 31 DOTs. The following are the findings from the survey:

- 1. 83.9% of the DOTs have constructed MSE abutments in their respective States while 16.1% reported on the contrary.
- 63.9% and 69.2% of the DOTs have constructed direct and mixed MSE abutments, respectively, with heights of 21–30 ft.
- 3. 50% of the DOTs use only precast concrete panels as the facing element for both direct as well as mixed MSE abutments.
- 4. Steel strips are the preferred choice of reinforcement by most DOTs (used by 40% of them) for both direct as well as mixed MSE abutments.
- 5. 46.4% of the DOTs reported the clear horizontal distance from the back of the wall facing to the front edge of the footing to be within 2 ft., while 39.3% of the DOTs reported it to be within 2.1–4.0 ft. The minimum requirement specified by FHWA (2009) is 0.5 ft.
- 6. 37.5% of the DOTs reported the depth of embedment of the footing to be between 1.1 and 2.0 ft. However, no guidelines have been specified by FHWA (2009) for the depth of embedment of the footing in a direct MSE abutment.
- 7. 46.2% of the DOTs reported the vertical clearance between the base of the footing and the topmost reinforcement layer to be within 0.6–1.0 ft. The minimum requirement specified by FHWA (2009) is 1 ft.
- 8. 50% of the piles used by DOTs in mixed MSE abutments are partial-displacement piles, 31.25% are displacement piles and 18.75% are non-displacement piles.
- 9. 78.1% and 21.9% of partial-displacement piles are H-piles and open-ended pipe piles, respectively, whereas 80% and 20% of displacement piles are closed-ended pipe piles and prestressed concrete piles, respectively. The non-displacement piles consist of drilled shafts.
- 10. 53.8% of the DOTs reported the clear horizontal distance from the back of the wall facing to the front edge of a driven pile to be between 2.1 and 4.0 ft. while 28.2% reported it to be within 4.1–6.0 ft. The minimum requirement specified by FHWA (2009) is 1.5 ft.
- 11. 61.5% of the DOTs reported the clear horizontal distance from the back of the wall facing to the front edge of a drilled shaft to be between 2.1 and 4.0 ft. while 23.1% reported it to be within 4.1–6.0 ft. The minimum requirement specified by FHWA (2009) is 3 ft.
- 12. The top five problems experienced by DOTs with respect to construction and performance of MSE abutments are

- a. loss of backfill material through the joints of the facing (18.1%);
- b. inadequate drainage system (12.5%);
- c. unsatisfactory workmanship and QA/QC (12.5%);
- d. growth of vegetation in the joints of the facing (11.1%); and
- e. inconsistent backfill compaction (11.1%).

7.2 Implementation Plan

The recommendations provided for the clear horizontal distance from the back of the wall facing to the front edge of the footing, the depth of embedment of the footing, and the length of the reinforcement, can be taken into account during design and construction of direct MSE abutments.

It should be noted that these recommendations are based on results obtained from two-dimensional finite element simulations performed by modelling the steel strips as sheet reinforcements with equivalent geometry and stiffness. The recommendations may be further improved by performing more sophisticated and complex three-dimensional finite element simulations that consider the discrete nature of the steel strips.

7.3 Scope for Further Study

- 1. The FE simulations may be extended to investigate the behavior of mixed MSE abutments in order to shed light on the response of the facing-backfill-pile systems along with pile group effects.
- 2. Field instrumentation of a direct and a mixed MSE abutment with geosynthetic or steel reinforcement could be performed in order to evaluate, monitor and assess their long-term performance. The resulting data would be very useful for design validation as well as for building confidence on the design and performance of MSE abutments for bridge support.

REFERENCES

- AASHTO. (2012). AASHTO LRFD bridge design specifications (6th ed.). Washington, DC: American Association of State Highway and Transportation Officials.
- ABAQUS. (2012). ABAQUS 6.12-2 [Software]. Providence, RI: SIMULIA, Inc.
- Abdelouhab, A., Dias, D., & Freitag, N. (2011). Numerical analysis of the behaviour of mechanically stabilized earth walls reinforced with different types of strips. *Geotextiles* and Geomembranes, 29(2), 116–129. https://doi.org/10.1016/ j.geotexmem.2010.10.011
- Abedi, S., Rechenmacher, A., & Chupin, O. (2010). Evolution of force chains in shear bands in sands. *Geotechnique*, 60(5), 343–351. https://doi.org/10.1680/geot.2010.60.5.343
- Abu-Hejleh, N., Wang, T., & Zornberg, J. G. (2000). Performance of geosynthetic-reinforced walls supporting bridge and approaching roadway structures. In J. C. Zornberg & B. R. Christopher (Eds.), Advances in transportation and geoenvironmental systems using geosynthetics (GSP 103; pp. 218–243). Reston, VA: American Society for Civil Engineers. https://doi.org/10.1061/40515(291)15

- Allen, T. M., & Bathurst, R. J. (2014). Design and performance of a 6.3 m-high, block-faced geogrid wall designed using K-stiffness method. *Journal of Geotechnical and Geoenvironmental Engineering*, 140(2). https://doi.org/10.1061/ (ASCE)GT.1943-5606.0001013
- Allen, T. M., Christopher, B. R., Elias, V. E., & DiMaggio, J. (2001). Development of the simplified method for internal stability design of mechanically stabilized earth (MSE) walls (Report No. WA-RD 513.1). Olympia, WA: Washington State Department of Transportation.
- Alshibli, K., & Sture, S. (1999). Sand shear band thickness measurements by digital imaging techniques. *Journal of Computing in Civil Engineering*, 13(2), 103–109. https://doi. org/10.1061/(ASCE)0887-3801(1999)13:2(103)
- AZDOT. (2009). *Bridge design guidelines*. Phoenix, AZ: Arizona Department of Transportation.
- Balunaini, U. (2009). Experimental study of the use of mixtures of sand and tire shreds as a geotechnical material (Doctoral dissertation). West Lafayette, IN: Purdue University.
- Balunaini, U., & Prezzi, M. (2010). Interaction of ribbedmetal-strip reinforcement with tire shred-sand mixtures. *Geotechnical and Geological Engineering*, 28(2), 147–163. https://doi.org/10.1007/s10706-009-9288-6
- Bathurst, R. J., Huang, B., & Allen, T. M. (2011). Load and resistance factor design (LRFD) calibration of steel grid reinforced soil walls. *Georisk*, 5(3–4), 218–228. https://doi. org/10.1080/17499518.2010.489828
- Bathurst, R. J., Nernheim, A., Walters, D. L., Allen, T. M., Burgess, P., & Saunders, D. D. (2009). Influence of reinforcement stiffness and compaction on the performance of four geosynthetic-reinforced soil walls. *Geosynthetics International*, 16(1), 43–59. https://doi.org/10.1680/gein. 2009.16.1.43
- Bathurst, R. J., Walters, D., Vlachopoulos, N., Burgess, P., & Allen, T. M. (2000). Full scale testing of geosynthetic reinforced walls. In J. C. Zornberg & B. R. Christopher (Eds.), Advances in transportation and geoenvironmental systems using geosynthetics (GSP 103; pp. 201–217). Reston, VA: American Society for Civil Engineers. https://doi.org/ 10.1061/40515(291)14
- Benigni, C., Bosco, G., Cazzuffi, D., & Col, R. D. (1996). Construction and performance of an experimental largescale wall reinforced with geosynthetics. In H. Ochiai, N. Yasufuku, & K. Omine (Eds.), Landmarks in earth reinforcement: Proceedings of the International Symposium on Earth Reinforcement, Fukuoka, Kyushi, Japan, 12–14 November 1996 (pp. 315–320). Rotterdam, The Netherlands: A. A. Balkema.
- Berg, R., & Vulova, C. (2007). Effects of pile driving through a full-height precast concrete panel faced, geogrid-reinforced, mechanically stabilized earth (MSE) wall. In H. Deaton & Y. Hashash (Eds.), *Case studies in earth retaining structures* (GSP 159). Reston, VA: American Society for Civil Engineers. http://dx.doi.org/10.1061/40903(222)3
- Bozozuk, M. (1978). Bridge foundation moves. *Transportation Research Record*, 678, 17–21.
- Brabant, K. (2001). Mechanically stabilized earth walls for support of highway bridges (Unpublished manuscript for UMASS Lowell-Course No. 14.533). University of Massachusetts Lowell.
- Christopher, B. R., Gill, S. A., Giroud, J. P., Juran, I., Mitchell, J. K., Schlosser, F., & Dunnicliff, J. (1990). *Reinforced soil structures—Volume 1: Design and construction guidelines* (Publication No. FHWA-RD-89-043), Washington, DC: Federal Highway Administration.

- Collin, J. G. (1986). *Earth wall design* (Doctoral dissertation). Berkeley, CA: University of California Berkeley.
- Cristelo, N., Felix, C., Lopes, M. L., & Dias, M. (2016). Monitoring and numerical modelling of an instrumented mechanically stabilized earth wall. *Geosynthetics International*, 23(1), 48–61. https://doi.org/10.1680/jgein.15.00032
- Damians, I. P., Bathurst, R. J., Josa, A., & Lloret, A. (2015). Numerical analysis of an instrumented steel-reinforced soil wall. *International Journal of Geomechanics*, 15(1). https:// doi.org/10.1061/(ASCE)GM.1943-5622.0000394
- Damians, I. P., Bathurst, R. J., Josa, A., Lloret, A., & Albuquerque, P. J. R. (2013). Vertical-facing loads in steelreinforced soil walls. *Journal of Geotechnical and Geoenvironmental Engineering*, 139(9), 1419–1432. http://dx.doi.org/ 10.1061/(ASCE)GT.1943-5606.0000874
- Elias, V. E., & Christopher, B. R. (1997). Mechanically stabilized earth walls and reinforced soil slopes: Design and construction guidelines (Publication No. FHWA-SA-96-071). Washington, DC: Federal Highway Administration.
- Elias, V. E., Christopher, B. R., & Berg, R. R. (2001). Mechanically stabilized earth walls and reinforced soil slopes: Design and construction guidelines (Publication No. FHWA-NHI-00-043). Washington, DC: Federal Highway Administration.
- FHWA. (2009). Design of mechanically stabilized earth walls and reinforced soil slopes—Volume I (Publication No. FHWA-NHI-10-024). Washington, DC: Federal Highway Administration, U.S. Department of Transportation.
- FHWA. (2012). Geosynthetic reinforced soil integrated bridge system interim implementation guide (Publication No. FHWA-HRT-11-026). McLean, VA: Federal Highway Administration.
- Gotteland, P., Gourc, J. P., & Villard, P. (1997). Geosynthetic reinforced structures as bridge abutments: Full scale experimentation and comparison with modelisations. In T. H. Wu (Ed.), *Proceedings of the International Symposium on Mechanicallly Stabilized Backfill* (pp. 25–34). Rotterdam, The Netherlands: A. A. Balkema.
- Helwany, S. M. B., Wu, J. T. H., & Kitsabunnarat, A. (2007). Simulating the behavior of GRS bridge abutments. *Journal* of Geotechnical and Geoenvironmental Engineering, 133(10), 1229–1240. https://doi.org/10.1061/(ASCE)1090-0241(2007) 133:10(1229)
- Hirakawa, D., Takaoka, H., Tatsuoka, F., & Uchimura, T. (2004). Deformation characteristics of geosynthetic retaining wall loaded on the crest. In J. B. Shim, C. Yoo, & H.-Y. Jeon (Eds.), *Proceedings of the 3rd Asian Regional Conference on Geosynthetics, Seoul, Korea* (pp. 240–247). Juniper, FL: Minerva TRI. Retrieved from http://www.geo synthetica.net/Uploads/GeoAsia04Hirakawa.pdf
- Huang, B., Bathurst, R. J., & Hatami, K. (2009). Numerical study of reinforced soil segmental walls using three different constitutive soil models. *Journal of Geotechnical and Geoenvironmental Engineering*, 135(10), 1486–1498. http://dx. doi.org/10.1061/(ASCE)GT.1943-5606.0000092
- Jones, C. J. F. P. (1996). *Earth reinforcement and soil structures.* London, UK: Thomas Telford Ltd.
- Keller, G., & Devin, S. (2003). Geosynthetic-reinforced soil bridge abutments. *Transportation Research Record*, 1819, 362–368. https://doi.org/10.3141/1819b-46
- Ketchart, K., & Wu, J. T. H. (1997). Performance of geosynthetic-reinforced soil bridge pier and abutment, Denver, Colorado, USA. In T. H. Wu (Ed.), *Proceedings* of the International Symposium on Mechanicallly Stabilized Backfill (pp. 101–116). Rotterdam, The Netherlands: A. A. Balkema.

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- Ling, H. I., & Liu, H. (2009). Deformation analysis of reinforced soil retaining walls-simplistic versus sophisticated finite element analyses. *Acta Geotechnica*, 4(3), 203–213. https://doi.org/10.1007/s11440-009-0091-6
- Loukidis, D., & Salgado, R. (2008). Analysis of the shaft resistance of non-displacement piles in sand. *Geotechnique*, 58(4), 283–296. https://doi.org/10.1680/geot.2008.58.4.283
- Loukidis, D., & Salgado, R. (2009). Modeling sand response using two-surface plasticity. *Computers and Geotechnics*, *36*(1–2), 166–186. https://doi.org/10.1016/j.compgeo.2008. 02.009
- Loukidis, D., & Salgado, R. (2012). Active pressure on gravity walls supporting purely frictional soils. *Canadian Geotechnical Journal*, 49(1), 78–97. https://doi.org/10.1139/t11-087
- Miyata, Y., & Bathurst, R. J. (2012). Analysis and calibration of default steel strip pullout models used in Japan. *Soils and Foundations*, 52(3), 481–497. https://doi.org/10.1016/j.sandf. 2012.05.007
- Miyata, K., & Kawasaki, H. (1994). Reinforced soil retaining walls by FRP geogrid. In F. Tatsuoka & D. Leshchinsky (Eds.), *Recent case histories of permanent geosyntheticreinforced soil retaining wall* (pp. 253–257). Rotterdam, The Netherlands: A.A. Balkema.
- Moulton, L. K., Ganga Rao, H. V. S., & Halvorsen, G. T. (1982). *Tolerable movement criteria for highway bridges* (Publication No. FHWA/RD-81/162). Washington, DC: Federal Highway Administration.
- Moulton, L. K., Ganga Rao, H. V. S., & Halvorsen, G. T. (1985). *Tolerable movement criteria for highway bridges* (Publication No. FHWA/RD-85/107). Washington, DC: Federal Highway Administration.
- Murthy, T. G., Loukidis, D., Carraro, J. A. H., Prezzi, M., & Salgado, R. (2007). Undrained monotonic response of clean and silty sands. *Géotechnique*, 57(3), 273–288. https://doi. org/10.1680/geot.2007.57.3.273
- Nelson, K. R. (2013). Lateral resistance of piles near vertical MSE abutment walls at Provo Center Street (Master's thesis). Provo, UT: Brigham Young University.
- Nemat-Nasser, S., & Okada, N. (2001). Radiographic and microscopic observation of shear bands in granular materials. *Géotechnique*, 51(9), 753–765. https://doi.org/10.1680/ geot.2001.51.9.753

- Runser, D. J., Fox, P. J., & Bourdeau, P. L. (2001). Field performance of a 17 m-high reinforced soil retaining wall. *Geosynthetics International*, 8(5), 367–391. https://doi.org/ 10.1680/gein.8.0200
- Salgado, R. (2008). *The engineering of foundations*. New York, NY: McGraw-Hill.
- Skinner, G. D., & Rowe, R. K. (2005). Design and behaviour of a geosynthetic reinforced retaining wall and bridge abutment on a yielding foundation. *Geotextiles and Geomembranes*, 23(3), 234–260. https://doi.org/10.1016/j.geotexmem. 2004.10.001
- Stuedlein, A. W., Bailey, M., Lindquist, D., Sankey, J., & Neely, W. J. (2010). Design and performance of a 46-mhigh MSE wall. *Journal of Geotechnical and Geoenvironmental Engineering*, 136(6), 786–796. https://doi.org/10.1061/ (ASCE)GT.1943-5606.0000294
- Wahls, H. (1983). Shallow foundations for highway structures (NCHRP Synthesis 107). Washington, DC: Transportation Research Board.
- Wu, J. T. H. (1992). Predicting performance of the Denver walls: General report. In *Proceedings of the International Symposium on Geosynthetic-Reinforced Soil Retaining Walls* (pp. 3–20). Rotterdamn, The Netherlands: A. A. Balkema.
- Yu, Y., Bathurst, R. J., & Allen, T. M. (2016). Numerical modeling of the SR-18 geogrid reinforced modular block retaining walls. *Journal of Geotechnical and Geoenvironmental Engineering*, 142(5). https://doi.org/10.1061/(ASCE) GT.1943-5606.0001438
- Yu, Y., Bathurst, R. J., & Miyata, Y. (2015). Numerical analysis of a mechanically stabilized earth wall reinforced with steel strips. *Soils and Foundations*, 55(3), 536–547. https://doi.org/10.1016/j.sandf.2015.04.006
- Zevgolis, L., & Bourdeau, P. L. (2007). Mechanically stabilized earth wall abutments for bridge support (Joint Transportation Research Program Publication No. FHWA/IN/JTRP-2006-38). West Lafayette, IN: Purdue University. https:// doi.org/10.5703/1288284313451
- Zheng, Y., & Fox, P. J. (2016). Numerical investigation of geosynthetic-reinforced soil bridge abutments under static loading. *Journal of Geotechnical and Geoenvironmental Engineering*, 142(5). https://doi.org/10.1061/(ASCE)GT.1943-5606.0001452

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