Design Method for Bearing Reinforcement Earth Wall

S. Horpibulsuk[1] , C. Suksiripattanapong[2] and A. Chinkulkijniwat[3]
School of Civil Engineering, Suranaree University of Technology, Nakhon Ratchasima, Thailand
[1]*E-mail*: suksun@g.sut.ac.th

ABSTRACT: The bearing reinforcement was developed as a cost-effective earth reinforcement. It is composed of a longitudinal member and transverse members. The longitudinal member is made of a deformed bar, which exhibits a high pullout friction resistance. The transverse members are a set of equal angles, which provide high pullout bearing resistance. The bearing reinforcement earth (BRE) walls have been applied as a bridge abutment and a retaining structure along mountainous areas in several projects of the Department of Highways, Thailand since 2008. Based on the laboratory and field studies and design experience, the design method of the BRE wall is presented. The examination of external stability is performed using the conventional method (limit equilibrium analysis) assuming that the composite backfill-reinforcement mass behaves as a rigid body. The internal stability deals with rupture and pullout resistances of the reinforcement. The pullout resistance of the bearing reinforcement is approximated using the modified punching shear mechanism. The maximum tension plane is the bilinear failure mechanism (coherent gravity structure hypothesis). Finally, a design procedure, which commonly used in Thailand, is summarized and suggested.

KEYWORDS: bearing reinforcement, inextensible reinforcement, mechanically stabilized earth wall, design method

1. INTRODUCTION

Soil reinforcing materials, such as strips and grids have been developed in the past two decades to increase the functional abilities for reinforced structures. In Thailand, a widely used strip reinforcement is the ribbed steel reinforcing strip (it is 50 mm in width and 4.2 mm in thickness with a yield strength of 520 MPa). This reinforcement is conveniently transported to a factory for galvanization and to a construction site as well as simple and fast to install due to its strip shape. Construction costs with strips are relative high because they are imported into Thailand from Africa and are subjected to high import charges. By contrast, steel grid reinforcements can be locally manufactured. The pullout resistance mechanisms of steel grid reinforcement have been extensively studied by many researchers (Peterson and Anderson, 1980; Jewell et al., 1984; Palmeira and Milligan, 1989; Palmeira, 2009; Bergado et al., 1988, 1996; Shivashankar, 1991; Chai, 1992; and Tin et al., 2011). The advantage of grid reinforcement is that the pullout bearing resistance in the resistant zone is high. However, the total volume (weight) of steel grid required is still high because of unnecessary transverse (bearing) bars in the active (unstable) zone. The transportation and installation of grid reinforcements are less convenient than those of strip reinforcements.

Horpibulsuk and Niramitkornburee (2010) have introduced a new cost-effective earth reinforcement designated as “Bearing reinforcement”. It is simple to install, convenient to transport and possesses high pullout and rupture resistances with less steel volume. Figure 1 shows the typical configuration of the bearing reinforcement, which is composed of a longitudinal member and transverse (bearing) members. The longitudinal member is a steel deformed bar and the transverse members are a set of steel equal angles. This reinforcement has been introduced into practice in Thailand since 2008 by the Geoform Co., Ltd. Several earth walls stabilized with the bearing reinforcements have been constructed in various parts of Thailand. The bearing reinforcement is connected to the facing panel at the tie point (2 U shape steel pieces) by a locking bar (a deformed bar) (Figure 2). This mechanically stabilized earth (MSE) wall is designated as “Bearing Reinforcement Earth (BRE) wall” (Horpibulsuk et al., 2010 and 2011). Figure 3 shows an example of a successful BRE wall as a highway structure in Saraburee Interchange project, Thailand.

For a BRE wall design, an examination of external and internal stability is a routine design procedure. The examination of external stability is generally performed using the conventional method (limit equilibrium analysis) assuming that the composite backfill-reinforcement mass behaves as a rigid body. The internal stability of the BRE wall deals with the rupture and pullout resistances of the reinforcement. During the past four years, the first author and their coworkers (Horpibulsuk and Niramitkornburee, 2010; Horpibulsuk et al., 2011; Suksiripattanapong et al., 2012 and 2013) investigated the interaction between soils and reinforcement, the BRE wall performance in comparison with the AASHTO’s recommendation and the numerical simulation of the BRE wall performance. This paper suggests a method for BRE wall design based on the research outcome. The design method has been successfully used to design several BRE walls in Thailand.

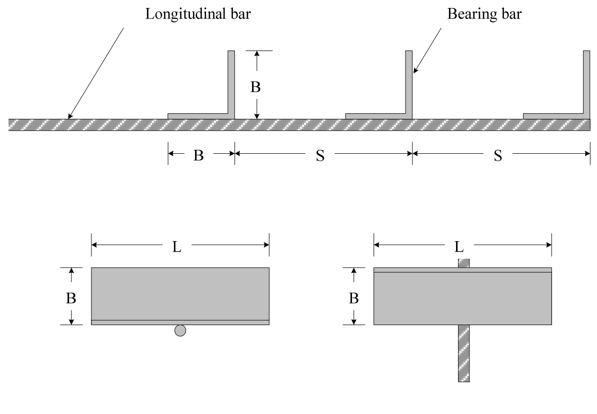

Figure 1 Configuration of the bearing reinforcement (Horpibulsuk and Niramitkornburee, 2010)

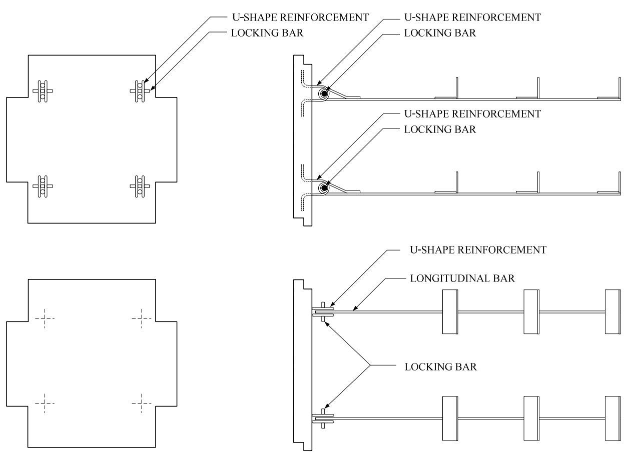

Figure 2 Connection of the bearing reinforcement to wall facing (Horpibulsuk and Niramitkornburee, 2010)

Figure 3 Application of BRE wall for highway structure in Suraburee Interchange project.

2. REINFORCED BACKFILL SOIL

Backfill is an important material affecting the pullout resistance of bearing reinforcement. The coarse-grained soils, which are not sensitive to change in moisture content, are recommended. Material to be used as a backfill soil must be tested and certified from a laboratory and must have the following properties.

- Liquid limit must not be greater than 30%.
- Plasticity index must not exceed 6%.
- Coefficient of uniformity must be greater than 4.
- pH as determined by AASHTO T-289 "Determination of soil for use in corrosion testing" must be between 5 and 10.
- Organic content as determined by AASHTO T-267 "Determination of organic content in soils by loss on ignition" must not exceed 1%.
- Friction angles as determined by AASHTO T-236 "Direct shear test of soils under consolidated drained conditions" should be greater than 32.
- Gradation of backfill for the bearing reinforcement is presented in Table 1.
- Electrochemical properties should be
  - o Soil resistivity as determined by AASHTO T-288 "Standard method for determining minimum laboratory soil resistivity" must not be less than 3000 Ωcm.
  - o Sulfates as determined by ASSHTO T-290 "Standard method for determining water-soluble sulfate ion content in soil" must not exceed 200 ppm.
  - o Chloride ion content in soil as determined by ASSHTO T-291 "Standard method for determining water-soluble chloride ion content in soil" exceeds 100 ppm.

If the resistivity is greater than or equal to 5,000 Ωcm, the chloride and sulfate requirements may be waived.

Table 1 Gradation of backfill for bearing reinforcement

Particle size (mm)	Percent passing (%)
37	100
4.75	30-100
0.425	15-100
0.150	5-65
0.075	0-15

3. EXTERNAL STABILITY

The BRE wall can be assumed as a rigid body for the examination of the external stability when the vertical spacing of the reinforcements is less than 800 mm (Horpibulsuk et al., 2010 and 2011). The embedded length of the bearing reinforcement of higher than 0.7 times the wall height is recommended (ASSHTO, 1996 and 2002).

Possible modes of failure of the BRE wall are illustrated in Figure 4, which are sliding, overturning, bearing and circular slip. The BRE wall has the external stability when no movement in three directions: horizontal, overturning, and vertical (bearing capacity). The horizontal and overturning stability were examined by the law of statics. The vertical movement was examined by the bearing capacity theory. For an examination of external stability, two cases of the live load are considered (*vide* Figure 5): 1) live load on both reinforced and unreinforced zones and 2) live load on unreinforced zone. The live load on reinforced zone increases stability against sliding and overturning but decreases stability against bearing failure. Case 2 is used to determine the factors of safety against sliding and overturning while the case 1 is used to determine the factor of safety against bearing failure. The conventional (limit equilibrium) method can be employed for this examination with the live load of about 20 kPa. This surcharge load is commonly taken for the MSE wall design in Thailand. AASHTO's Standard Specifications Highway Bridge Section 5.8 recommends that the factors of safety against sliding, overturning, and bearing failure should be greater than 1.5, 2.0, and 2.5, respectively.

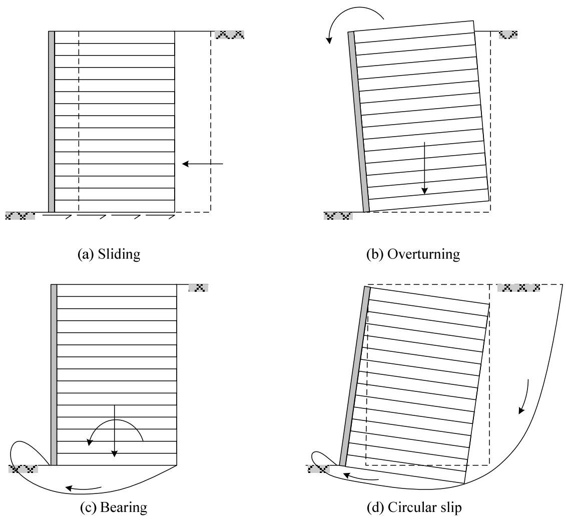

Figure 4 External stability of BRE wall

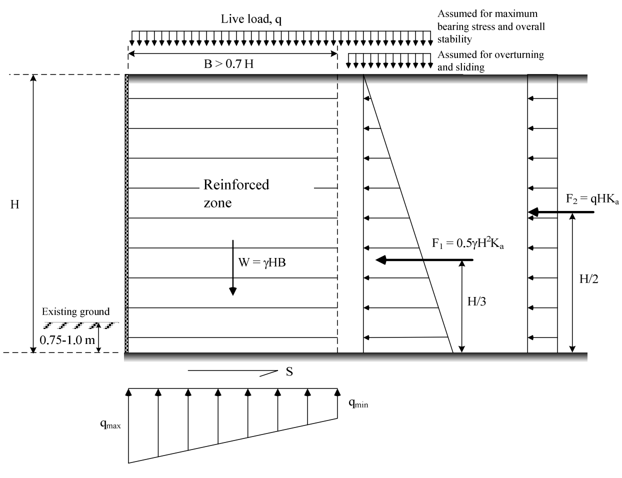

Figure 5 Forces acting on the BRE wall

4. INTERNAL STABILITY

The internal stability analysis deals with rupture and pullout mechanisms. Figure 6 summarizes these two possible modes of failure. The determination of the internal stability against rupture and pullout failures is being presented.

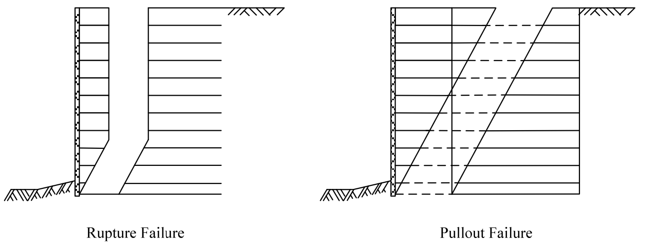

Figure 6 Internal stability of BRE wall

4.1 Stability against pullout failure

The factor of safety against pullout failure is determined by the ratio of the pullout resistance in the resisting zone to the maximum pullout force in the reinforcement. As such, three parameters required for the determination of the factor of safety are failure plane, maximum pullout force on the reinforcement and pullout resistance. Because it was impossible to determine the failure plane of the test BRE wall, the maximum tension plane obtained from the strain gauge is assumed as a possible failure plane. The maximum tension plane obtained from the full-scale test (Horpibulsuk et al., 2010 and 2011 and Suksiripattanapong et al., 2012) is shown in Figure 7. The possible failure plane (maximum tension plane) can be approximated from the coherent gravity structure hypothesis.

The maximum tension force is calculated from the $K\sigma_v$, where K is the coefficient of lateral earth pressure and σ_v is the normal stress. Based on the back analysis of the test data (*vide* Figure 8), the lateral earth pressure, σ_h, at each reinforcement level is approximated using $K = K_0$ at the top of the wall and decreases linearly to $K = K_a$ at 6 m depth. Below a 6 m depth, $K = K_a$ is used. The K at the wall face is lower than that at the maximum tension. It is slightly higher than K_a for the shallow depth and is close to K_a for the depth below 3 m.

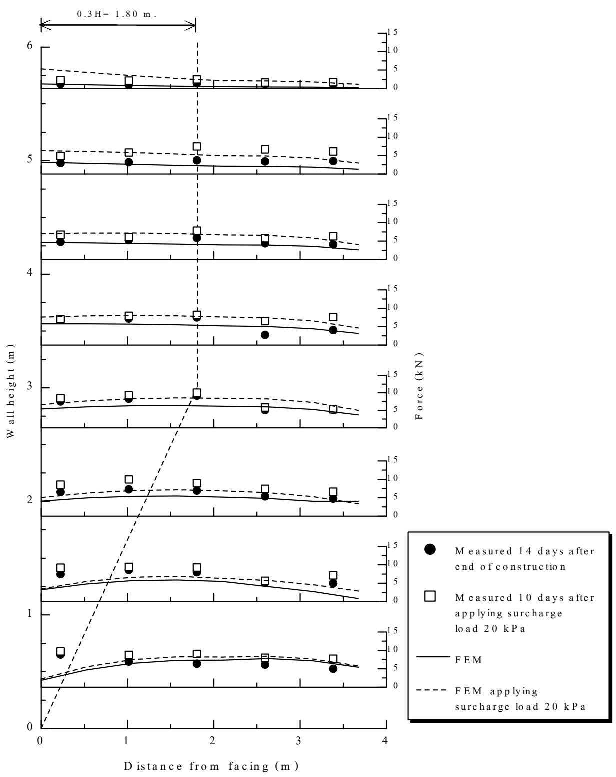

Figure 7 Maximum tension plane obtained from test data and numerical simulation (Suksiripattanapong et al., 2012)

From Figures 7 and 8, the failure plane and the K value can be approximated by the coherent gravity method, which are the typical characteristics of the inextensible reinforcement earth wall. Following presents the development of the pullout resistance equations in coarse grained soils. The pullout resistance, P_t of the bearing reinforcement is contributed from the pullout friction from the longitudinal member, P_f and bearing resistance from the transverse members, P_{bn}.

$$P_t = P_f + P_{bn} \tag{1}$$

Maximum pullout friction resistance, P_f, of the longitudinal member can be calculated from

$$P_f = \left(c_a + \sigma_v \tan\delta\right) D L_e \tag{2}$$

where c_a is the adhesion, δ is the skin friction angle, D is the diameter of the longitudinal member, L_e is the length of the longitudinal member in resistance zone and σ_n is the normal stress. Horpibulsuk and Niramitkornburee (2010) and Suksiripattanapong et al. (2013) have studied pullout of a deformed bar embedded in four different coarse-grained soils, having friction angles between 40 and 45 degrees and average grain sizes, D_{50} between 0.37 and 7 mm. The failure envelopes of all tested soils are shown in Figure 9. The shear stress, τ was determined from $P_{f\max}/\pi DL$ where L is the length of longitudinal member.

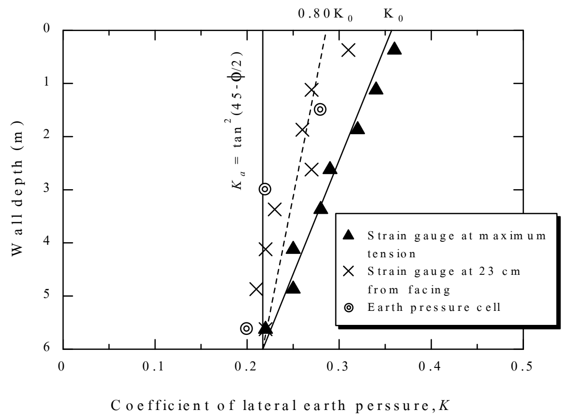

Figure 8 Coefficient of lateral earth pressure for the bearing reinforcement (Horpibulsuk et al., 2011)

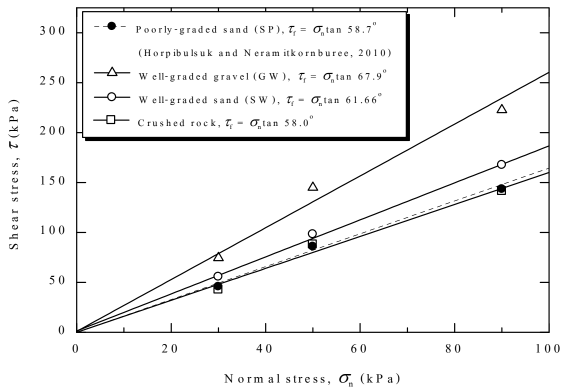

Figure 9 Failure envelope of different soils (Suksiripattanapong et al., 2013)

The δ values are very high and larger than friction angle, ϕ for all tested soils because the roughness of the deformed steel bar increases the failure friction plane during pullout (failure friction diameter is greater than the measured diameter of longitudinal member) and the concentration of vertical stresses on the rigid metal bar due to arching effects as a consequence of the higher stiffness of the bar in comparison to that of the soil (Suksiripattanapong et al., 2013).

It is now to present an equation to determine the pullout resistance of the transverse members. The bearing reinforcement consists of several transverse members placed at regular intervals. During the pullout of the bearing reinforcement, the transverse members interfere with each other. The pullout bearing resistance is thus calculated as follows:

$$P_{bn} = nFP_{b1} \tag{3}$$

where n is number of transverse members, F is the interference factor and P_{b1} is the pullout bearing of a single transverse member (no transverse member interference).

Based on the available research on the pullout bearing mechanisms of different reinforcement types (Alfaro et al., 1995; Hayashi et al., 1999; Alfaro and Pathak, 2005; AASHTO, 2002; Bergado et al., 1988, 1996; Shivashankar, 1991; Chai, 1992; Khedkar and Mandal, 2009; and Abdi and Arjomand, 2011), three existing pullout bearing failure mechanisms for the plane strain condition are proposed: general shear failure (Peterson and Anderson, 1980), punching shear failure (Jewell et al., 1984), and modified punching shear failure (Chai, 1992; Bergado et al., 1996; Horpibulsuk and Niramitkornburee, 2010 and Suksiripattanapong et al., 2013). The maximum bearing stress of a single isolated transverse member, $\sigma_{b\max}$, in coarse-grained soil is presented in the form:

$$\sigma_{b\max} = N_q \sigma_n \tag{4}$$

where N_q is bearing capacity factor, depending upon the mode of failure and σ_n is normal stress. N_q for the three failure mechanisms is presented in terms of soil friction angle, ϕ, as follows:

$$N_{q(\text{general})} = \exp\left[\pi \tan\phi\right]\tan^2\left(\frac{\pi}{4}+\frac{\phi}{2}\right)$$
for general shear failure (5)

$$N_{q(\text{punching})} = \exp\left[\left(\frac{\pi}{2}+\phi\right)\tan\phi\right]\tan\left(\frac{\pi}{4}+\frac{\phi}{2}\right)$$

for punching shear failure (6)

$$N_{q(\text{modified})} = \frac{1}{\cos\phi}\exp\left[\pi \tan\phi\right]\tan\left(\frac{\pi}{4}+\frac{\phi}{2}\right)$$
for modified punching shear failure (7)

Using the proposed equations (Eqs.4 to 7), the comparison between the measured and predicted maximum bearing stresses are shown in Figure 10 by Suksiripattanapong et al. (2013) to recommend the practical N_q value for examining the internal stability. For the well-graded gravel (GW) and the crushed rock (GP), with large average grain size, D_{50}, the maximum bearing stress, $\sigma_{b\max}$, at low normal stress of about 30 kPa was close to that predicted by the general shear mechanism. But the measured maximum bearing stress, $\sigma_{b\max}$, at high normal stress of 90 kPa was very close to the predicted one by the modified punching shear mechanism. The same is not for the small D_{50} soils (SP and SW). The measured pullout bearing stress is predicted satisfactorily based on the modified punching shear mechanism for different normal stresses. As the bearing reinforcement is pulled out and shear displacement occurs along the interface, the zone of soil surrounding the reinforcement tends to dilate. However, the volume change is restrained by the surrounding non-dilating soil, resulting in an increase in normal stress on the soil-reinforcement interface (interlocking effect). The interlocking effect is significant for the large particle soils and can be ignored for the small particle soils. Hence, the pullout mechanism of the bearing reinforcement embedded in the gravelly soils (both well-graded and poorly graded) under low normal stress approaches the general shear failure. This effect decreases as the increase in the normal stress. To conclude, due to the effect of interlocking, the pullout resistance is larger than that approximated from the modified punching shear mechanism (Eq. 7). This effect exists only when the B/D_{50} is less than 12 and the normal stress is less than 90 kPa. For the conservative design, the pullout resistance can be approximated based on the modified punching shear mechanism.

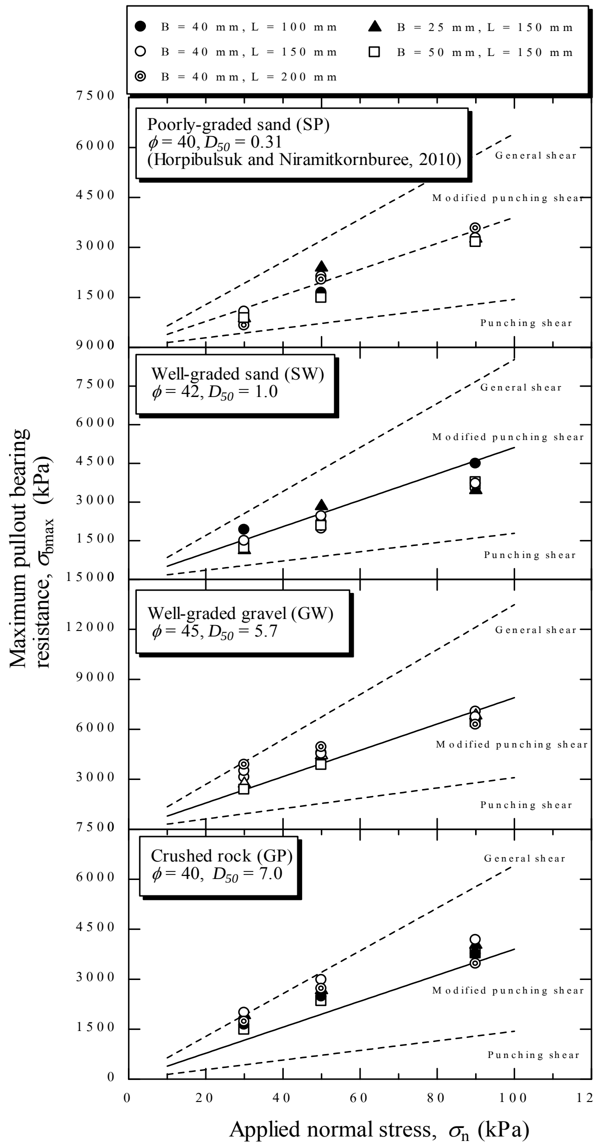

Figure 10 Maximum pullout bearing resistance of a single isolated transverse member (Suksiripattanapong et al., 2013)

A dimensionless parameter, transverse member spacing ratio, S/B was introduced to investigate the influence of spacing, S, and dimension (B and L) of transverse members on the pullout bearing characteristics (Horpibulsuk and Niramitkornburee, 2010 and Suksiripattanapong et al., 2013). Generally, the larger the S/B, the higher the pullout bearing resistance up to a certain maximum value, due to less interference among transverse members. Similarly, for steel grid, the transverse member interference is dependent upon the spacing of transverse member and the diameter of transverse members (Bergado and Chai, 1994 and Bergado et al., 1996).

Suksiripattanapong et al. (2013) illustrated the effect of S/B on the maximum pullout bearing force, P_{bn} of bearing reinforcement embedded in different soils with large and small grains as presented in Figure 11. It is for 40x150 mm transverse members (n = 2 to 4) under different applied normal stresses. The maximum pullout bearing force, P_{bn} is compared with maximum pullout bearing force of a single isolated transverse member (n = 1), P_{b1}. The failure mechanism of the bearing reinforcement is classified into three zones, depending upon the S/B value. Zone 1 is referred to as block failure when the $S/B \le 3.75$. Zone 2 is regarded as member interference failure when $3.75 < S/B < 25$. Zone 3 ($S/B > 25$) is individual failure where soil in front of each transverse member fails individually. The interference factor, F was proposed as follows:

$$F = a + b\ln\left(\frac{S}{B}\right) \tag{8}$$

where a and b are constant, depending upon n. These two constants can be obtained with the two physical conditions: 1) when S/B equals 3.75, the interference factor equals $1/n$ since P_{bn} and P_{b1} are the same, and 2) when S/B equals 25, the interference factor equals unity. These two conditions establish the lower and upper values of F at corresponding values of S/B = 3.75 and 25, respectively. From these two conditions, the constants a and b can be determined by the following equations:

$$b = 0.527\left[1 - \frac{1}{n}\right] \tag{9}$$

$$a = 1 - 3.219b \tag{10}$$

It is of interest to conclude that the member interference is dependent on only the S/B, irrespective of grain size distribution and friction angle.

4.2 Stability against pullout failure

The factor of safety against rupture failure is the ratio of rupture strength to the maximum tension force. This maximum tension forces is assumed to be equal to the maximum pullout force. The rupture strength is the product of the yield stress and cross sectional area of the longitudinal member at a design life. The cross sectional area is approximated from the reinforcement corrosion as follows (AASHTO, 2002):

Galvanized loss

- 15 micron (0.015 mm) per year at first 2 years
- 4 micron (0.004 mm) per year in subsequent years

Steel loss

- 12 micron (0.012 mm) per year after zinc depletion for the remaining years until design life

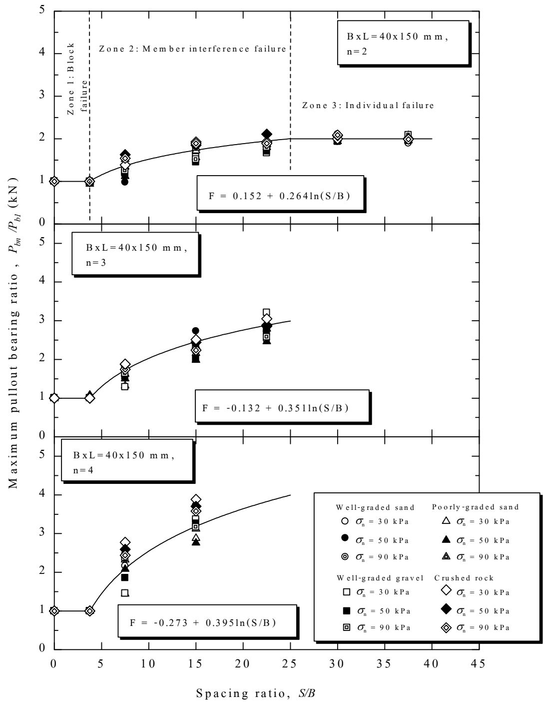

Figure 11 Measured and predicted P_{bn}/P_{b1} and S/B relationship for 40x150 mm transverse members(Suksiripattanapong et al., 2013)

5. SUMMARY OF DESIGN PROCEDURES

In Thailand, the MSE walls of the Department of Highways are generally required to found on the hard stratum. The suggested design method for the BRE wall founded on the hard stratum is being presented. It has been successfully used for designing several BRE walls under the supervision of the Department of Highways, Thailand. The built BRE wall projects in Thailand include the Northern Saraburi Interchange; the Highway Bridge, Highway No. 418; and the Highway Route No.4 Phathalung-Trang, etc. The BRE wall was designed by examining the external and internal stabilities. The BRE wall can be assumed as a rigid body when the vertical spacing of the reinforcements is less than 800 mm for the examination of the external stability. The embedded length of the bearing reinforcement of higher than 0.7 times the wall height is recommended (ASSHTO, 1996 and 2002). The conventional (limit equilibrium) method can be employed for this examination with the live load of about 20 kPa. A suggested procedure for examining the stability of the BRE wall is proposed as follows:

Determine the maximum pullout force in the bearing reinforcement

1. Based on the coherent gravity structure hypothesis, approximate the maximum tension (possible failure) plane for the designed BRE wall.
2. Determine the maximum pullout forces in the bearing reinforcements by multiplying the vertical stress by the coefficient of lateral earth pressure, K, and the vertical and horizontal spacing (S_v and S_h) of the bearing reinforcement. The relationship between K and depth presented in Figure 8 is recommended for this calculation.

Determine the rupture strength of the bearing reinforcement

3. Perform a tensile test on the longitudinal member to determine the yield strength.
4. Determine the rupture strength of the longitudinal member by multiplying the yield strength by the cross-sectional area at the design life. The cross-sectional area is approximated using the AASHTO's recommendation (section 4.2).

Determine the pullout resistance of the bearing reinforcement

5. Perform a large direct shear test on the backfill material to determine shear strength parameters.
6. Determine apparent δ for the friction pullout resistance, which can be directly obtained from a pullout test on a longitudinal member or approximated from apparent δ / ϕ = 1.0 for conservative design.
7. Determine total pullout resistance P_t, which is the sum of the friction and bearing pullout resistances using Eqs. (3), (7)-(10). The effect of corrosion on the area of the longitudinal and transverse members must be considered.

Examine the internal stability

8. Determine the factor of safety against rupture failure. This factor of safety must be greater than 2.0.
9. Determine the factor of safety against pullout failure. This factor of safety must be greater than 1.5.

Examine the external stability

10. Determine the factor of safety against sliding, overturning and bearing failures. The factors of safety must be greater than 1.5, 2.0 and 2.5, respectively. The live load of 20 kPa is recommended for the BRE design in Thailand.

6. DESIGN EXAMPLE OF THE BRE WALL

A design example is presented in this section. As shown in Figure 5, the height of BRE wall, H is 6 m. The longitudinal members are 12 mm diameter and 4.2 m length with the yield strength of 400 MPa. The BRE wall has 8 layers of reinforcement. The vertical spacing, S_v is 750 mm. The horizontal spacing, S_h is 750 and 500 mm for 4th to 8th (top) and 1st (bottom) to 3rd reinforcement layers, respectively. The transverse members are 25 mm leg length (B) and 180 mm length (L). The spacing between equal steel angles (S) is 750 mm, which is larger than 25B, hence no transverse member interference. The backfill and foundation properties are shown in Table 2.

Table 2 The backfill and foundation properties

Item	Backfill	Foundation
$\gamma_{d\,max}$	16.9 kN/m^3	17 kN/m^3
ϕ	40^o	34^o

6.1 Examination of external stability

The calculation is divided in two cases (as explained in section 3): live load on both reinforced and unreinforced zones and live load on the unreinforced zone.

- Live load on both reinforced and unreinforced zones
  - Factor of safety against overturning

$$\mathrm{FS}_{\mathrm{overturning}} = \frac{\sum \text{resisting moments}}{\sum \text{overturning moments}} > 2.0$$

$$\mathrm{FS}_{\mathrm{overturning}} = \frac{W(4.2/2)}{F_1(H/3) + F_2(H/2)}$$

$$\mathrm{FS}_{\mathrm{overturning}} = \frac{894.3}{132 + 78.1} = 4.26 > 2.0$$

  - Factor of safety against sliding

$$\mathrm{FS}_{\mathrm{sliding}} = \frac{\sum \text{sliding resistance}}{\sum \text{sliding forces}} > 1.5$$

$$\mathrm{FS}_{\mathrm{sliding}} = \frac{W \tan\delta}{F_1 + F_2} > 1.5$$

$$\mathrm{FS}_{\mathrm{sliding}} = \frac{287.07}{66 + 26} = 3.12 > 1.5$$

  - Factor of safety against bearing failure

$$\mathrm{FS}_{\mathrm{bearing}} = \frac{q_{ult}}{q_{av}} = \frac{0.5\gamma(B-2e)N_\gamma}{W/(B-2e)} > 2.5$$

$$N_\mathrm{q} = e^{\pi\tan\phi}\tan^2\left(45^\circ + \frac{\phi}{2}\right) = 29.3$$

$$N_\gamma = \left(N_q - 1\right)\tan\left(1.4\phi\right) = 31$$

$$e = \frac{B}{2} - \left(\frac{\sum M_r - \sum M_o}{\sum V}\right) < \frac{B}{6}$$

$$e = \frac{4.2}{2} - \left(\frac{894.3 - 132}{425.9}\right) = 0.31 < 0.7$$

Therefore

$$\mathrm{FS}_{\mathrm{bearing}} = \frac{0.5(17)\left(4.2 - 2(0.31)\right)31}{894.3/\left(4.2 - 2(0.31)\right)} = 7.9 > 2.5$$

- Live load on unreinforced zones
  - Factor of safety against overturning

$$\mathrm{FS}_{\mathrm{overturning}} = \frac{894.3 + 176.4}{132 + 78.1} = 5.1 > 2.0$$

  - Factor of safety against sliding

$$\mathrm{FS}_{\mathrm{sliding}} = \frac{343.7}{66 + 26} = 3.73 > 1.5$$

  - Factor of safety against bearing failure

$$\mathrm{FS}_{\mathrm{bearing}} = \frac{q_{ult}}{q_{av}} = \frac{0.5\gamma(B-2e)N_\gamma}{\left(W + LL(B)\right)/(B-2e)} > 2.5$$

$$e = \frac{B}{2} - \left(\frac{\sum M_r - \sum M_o}{\sum V}\right) < \frac{B}{6}$$

$$e = \frac{4.2}{2} - \left(\frac{894.3 + 176.4 - 132 - 78.1}{425.9 + (20 * 4.2)}\right) = 0.41 < 0.7$$

Therefore

$$\mathrm{FS}_{\mathrm{bearing}} = \frac{0.5(17)\left(4.2 - 2(0.41)\right)31}{\left(894.3 + 20\times 4.2\right)/\left(4.2 - 2(0.41)\right)} = 5.9 > 2.5$$

6.2 Examination of internal stability

Table 3 shows the calculated maximum pullout force per reinforcement, $T_{\max}$, total pullout resistance, P_t, factors of safety against rupture, FS_{rup}, and factors of safety against pullout failure, FS_{rup} for each reinforcement layer. In this calculation, the δ/ϕ ratio was assumed as 1.0 and no corrosion was considered. For conservative, K_0 was used to determine $T_{\max}$ for all reinforcement layers. Eq. (7) was used to determine bearing capacity factor N_q.

Table 3 Internal stability examination for the test BRE wall

Layer	Z (m)	n	L_e	T_{max} (kN)	N_q	P_f (kN)	P_{bn} (kN)	P_t (kN)	FS_{rup}	FS_{pull}
8	0.375	3	2.4	5.3	39.1	2.7	13.9	16.6	8.4	3.1
7	1.125	3	2.4	7.9	39.1	3.9	20.6	24.5	5.7	3.1
6	1.875	3	2.4	10.4	39.1	5.2	27.3	32.5	4.3	3.1
5	2.625	3	2.4	13.0	39.1	6.5	34.0	40.5	3.4	3.1
4	3.375	3	2.6	15.5	39.1	8.5	40.7	49.2	2.9	3.2
3	4.125	2	3.1	12.1	39.1	17.4	31.6	49.0	3.7	4.1
2	4.875	2	3.5	13.7	39.1	22.8	36.0	58.8	3.2	4.3
1	5.625	2	4.0	15.4	39.1	28.9	40.5	69.4	2.88	4.5

7. CONCLUSION

This paper introduces a new mechanically stabilized earth wall in Thailand, designated as a bearing reinforcement earth (BRE) wall. The bearing reinforcement is a cost-effective earth reinforcement, which is composed of a longitudinal member and transverse members. The longitudinal member is made of a deformed bar, which exhibits a high pullout friction resistance. The transverse members are a set of equal angles, which provide high pullout bearing resistance. Based on the extensively study during the past four years, the design method of the BRE wall is suggested in this paper. The design deals with the examination of external and internal stability. The BRE wall can be assumed as a rigid body when the vertical spacing of the reinforcements is less than 800 mm. The possible failure plane (maximum tension plane) and the maximum tension force can be approximated from the coherent gravity structure hypothesis. The pullout resistance of the bearing reinforcement is calculated in terms of number of transverse members, interference factor and bearing capacity factor. The bearing capacity based on the modified punching shear mechanism is recommended in practice. The suggested design method has been successfully used to design several BRE wall in Thailand.

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