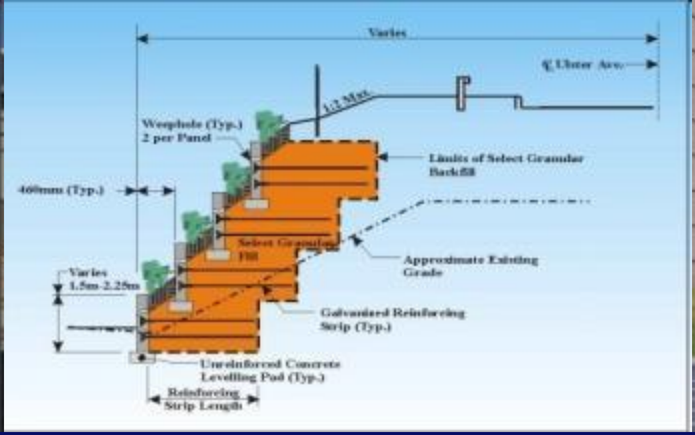

Proposed Modifications of the K-Stiffness Method for MSE Structures on Soft Ground

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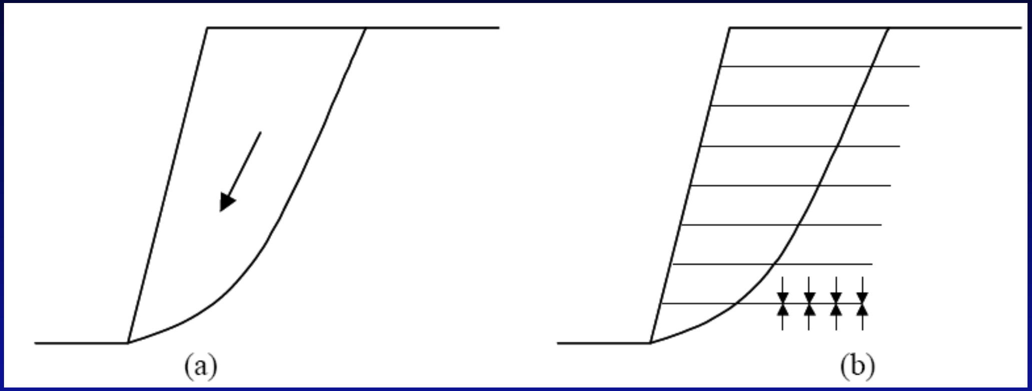
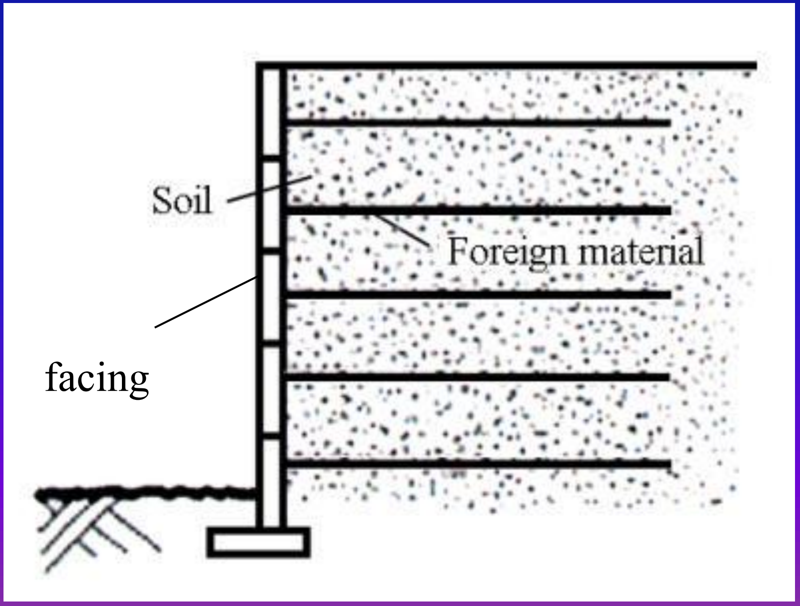
Introduction



Unreinforced slope reinforced soil slope

(MSE) structures have been effective alternatives for many applications to retain walls by adding foreign material to strengthen the soil.



Three components of MSE structures:

- Filling materials
- Reinforcements
- Face elements

Components of MSE structures

- **Filling Materials**:
  - frictional soil: good drainage, mobilize the friction between soil and reinforcement → encouraged to be used
  - cohesive soil: poor drainage → sensitivity with moisture content changes
  - cohesive-friction soil
  - lightweight geomaterials (rubber sand) → reducing the weight of structure on the foundation
- **Reinforcement Material** :
  - Inextensible reinforcement: hexagonal wire mesh, steel strip, welded wire, steel grid
  - Extensible reinforcement: geosynthetics
- **Facing**:
  - Flexible wall
  - Stiff wall

Failure modes of MSE walls

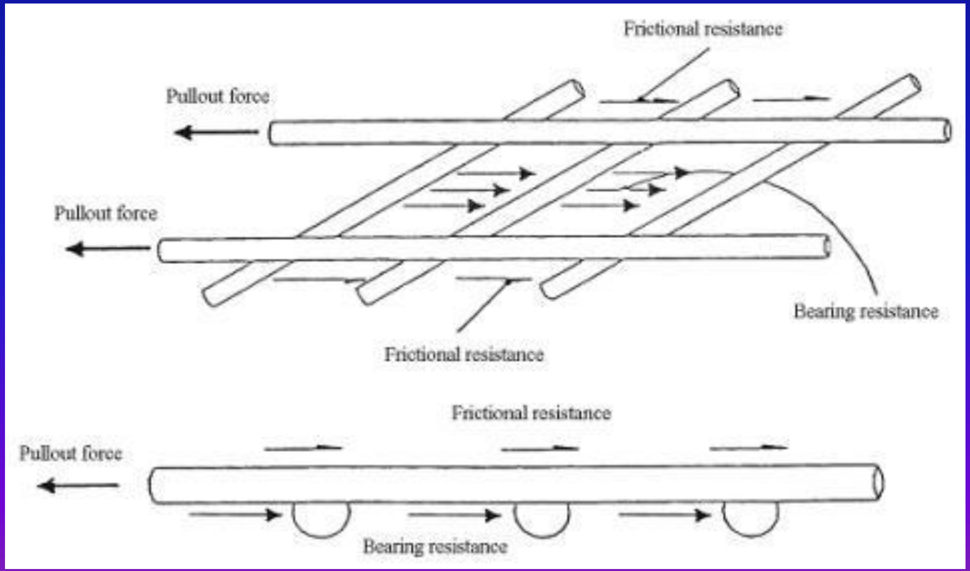
Internal failure:

- ✓ Tension failure: the tension in the reinforcement layers exceeds its tensile strength → rupture of reinforcement

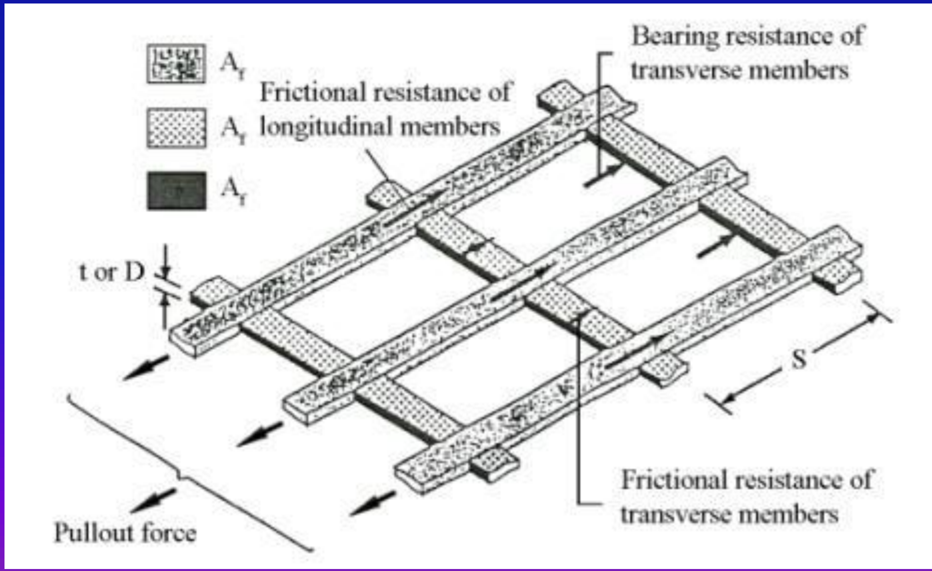
- ✓ Slippage (pullout) failure: tension is less then tension strength but greater than pullout resistance of the reinforcement → slippage between soil and reinforcement

Pullout resistance of the grid reinforcement:

frictional resistance and or bearing resistance



steel grid reinforcement



geogrid reinforcement

Pullout resistance of the reinforcement

Frictional resistance

$$P_f = A_s x \overline{\sigma_s} x \tan\delta$$

A_s: frictional area between soil and grid reinforcement

$\overline{\sigma_s}$ = average normal stress (equal to $0.75\sigma_v$ for inextensible grid reinfo rcement)

δ = skin friction angle between soil and grid reinforcement

steel grid reinforcement	geogrid reinforcement
- surface area of the longitudinal ribs - about 10% of pullout resistance (Abiera, 1991)	- surface area of the longitudinal ribs and the transverse bars - about 90% of pullout resistance (Abiera, 1991)

Bearing resistance: only on the areas of grid transverse members

$$P_p = \overline{\sigma_b} xnxd$$

$\overline{\sigma_b}$ = maximum bearing stress against single transverse members

n = number of transverse members

d = diameter or width of a single transverse member being normal to the maximum bearing stress.

Current design methods used to calculate reinforcement loads in MSE structures

Simplified Method

(Using limit equilibrium concepts to develop the design model)

$$T_{max} = S_v K_a (\gamma[z+S]+q)$$

S_v = tributary area for reinforcement layer

K_a = coefficient of active earth pressure, determined with a horizontal backslope and no wall-soil interface friction

γ = unit weight of the soil

z = depth of reinforcement layer below the top of the wall

S = equivalent soil height of uniform surcharge pressure.

q = surcharge pressure

Current design methods used to calculate reinforcement loads in MSE structures

FHWA Structure Stiffness Method

(Using limit equilibrium concepts to develop the design model)

$$T_{\max} = S_v K_r (\gamma[z+S]+q)$$

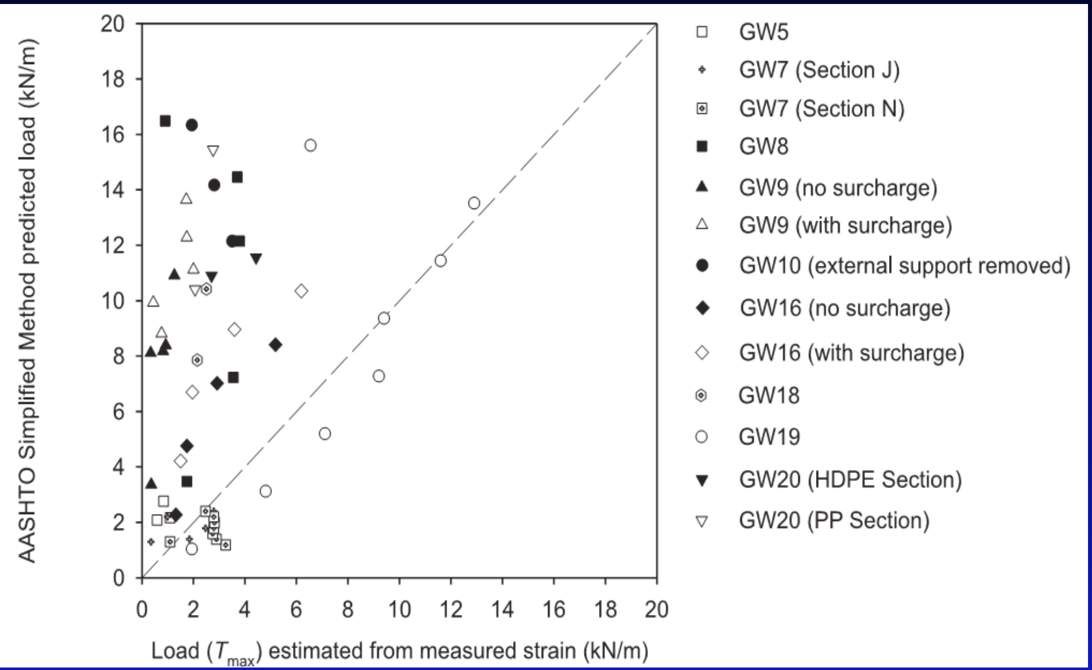
$$K_r = K_a(\Omega_1(1+0.4\frac{S_r}{47{,}880})(1-\frac{z}{6})+\Omega_2\frac{z}{6}) \quad \text{if } z\ (m) \le 6m$$

$$K_r = K_a\Omega_2 \quad \text{if } z\ (m) > 6m$$

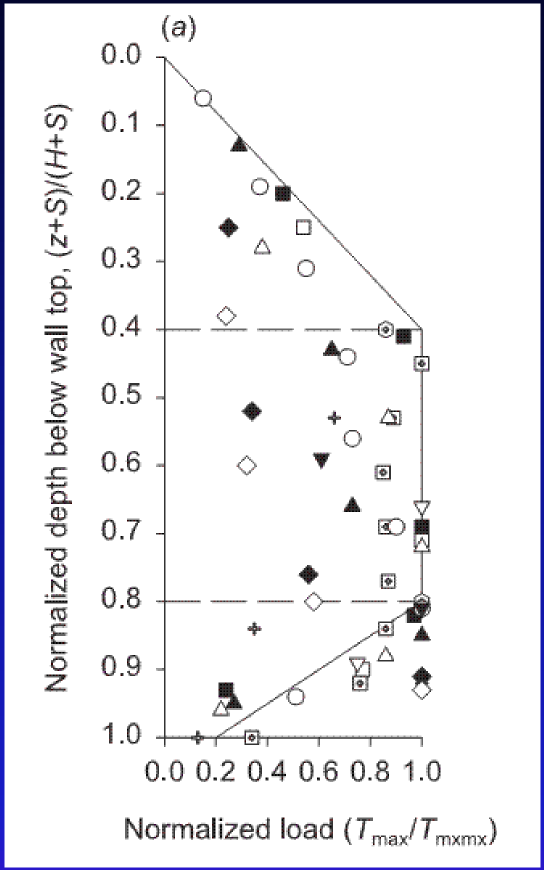
$$S_r = \frac{J}{(\frac{H}{n})}$$

- K_r = lateral earth pressure coefficient
- S_r = global reinforcement stiffness for the wall
- Ω_1 = 1.0 for strip and sheet reinforcement or 1.5 for geogrid and welded wire mats.
- Ω_2 = 1.0 if Sr ≤ 47,880 kPa or Ω 2 = Ω1 if Sr > 47,880 kPa.
- J = average reinforcement stiffness for the wall

Comments on Current Design Method



- a large amount of scatter
- the predicted loads were greater than the estimated loads



The load distribution envelope was rather trapezoidal in shape, not triangular as it was assumed for design

Methods used to calculate reinforcement loads in MSE structures

Sources of conservatism

- stiffness of various wall components and toe restraint were not explicitly considered in the ASSHTO Simplified Method

- using laboratory shear strength values that are not corrected for the plane strain conditions

- the assumption that the wall is at a state of limit equilibrium → the strength of the soil and the reinforcement is fully mobilized everywhere and all wall components of the wall are at a state of incipient collapse

≠ reinforcement loads estimated from measured strains: at working stress conditions

The reinforcement loads do not represent the soil state of stress:

+ the force in the reinforcement only depends on the strain and the stiffness of the reinforcement

+ shear stress occurring at the soil/ reinforcement interface ⇒ equating the soil stress (K_a or K_0) to the reinforcement load which assumes that principle stress direction remains vertical and horizontal is not reasonable

Back

K-stiffness method

Allen and Bathurst (2002b): $J_i = J_{2\%}$

(1) prevent failure of the reinforced soil (i.e., to avoid failure of the soil as a limit state for internal design of reinforced soil walls)

(2) Good performance of walls with granular backfill defined by acceptable post-construction outward wall deformation and no cracking at the surface of the reinforced soil zone behind the wall facing was achieved with typically recorded strain less than 2% at end of construction

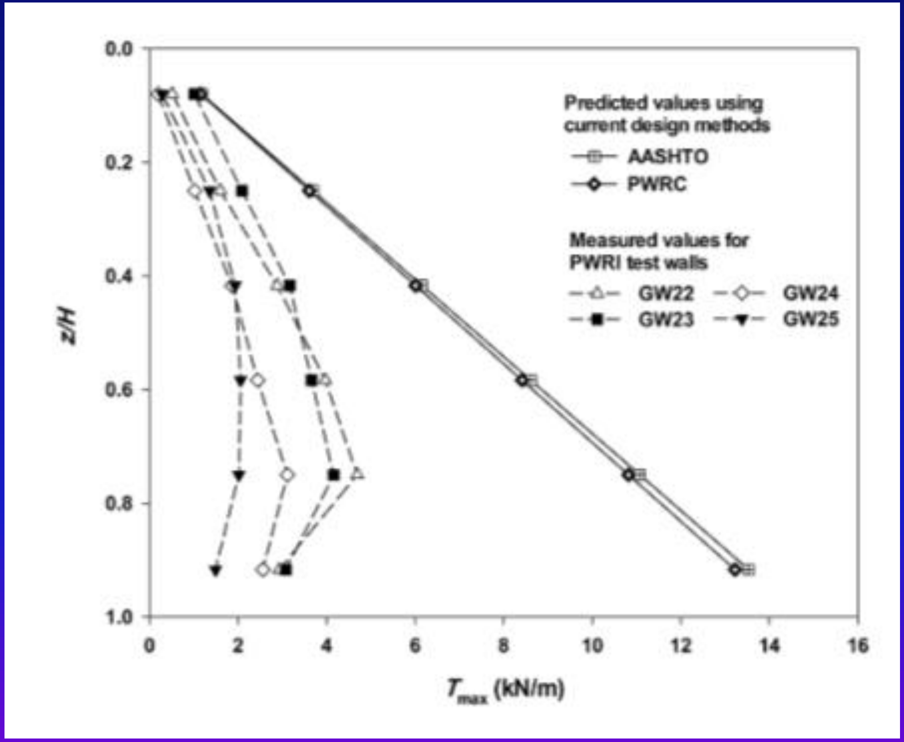
(3) Creep strains and strain rates were observed to decrease as time increases (i.e., only primary creep occurs) when end of construction reinforcement strains were less than 2%.

Back

Working stress condition

North American working stress design practice: factors of safety have been assigned to failure modes such as external, internal or facing stability.

Some issues of current working stress design for geosynthetic reinforced soil retaining walls (Bathurst 2008):



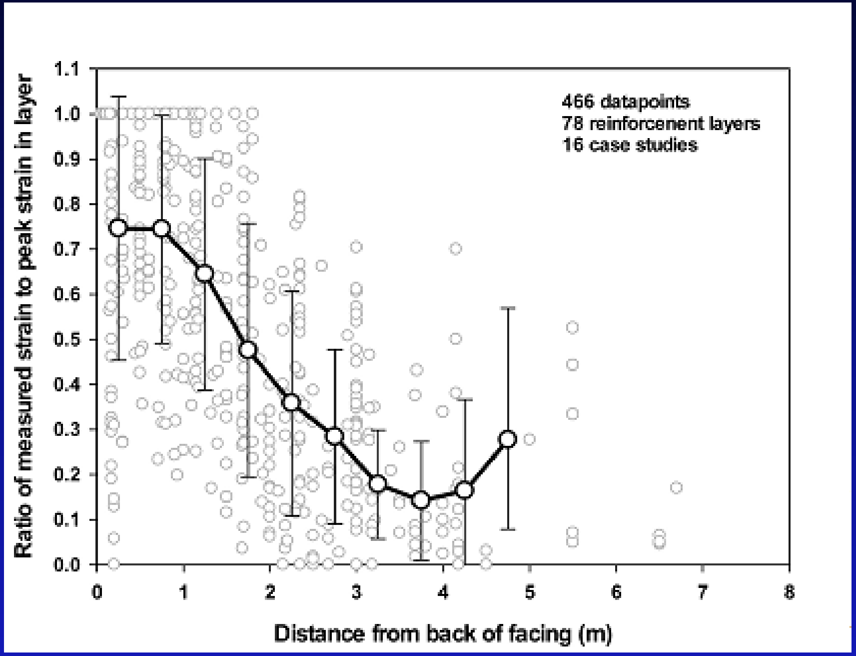
The stresses at incipient collapse could not be simply considered to be the scaling of failure loads and resistance at limit equilibrium to working stress conditions using one or more factors of safety or partial factors

Predicted versus measured values of Tmax

Working stress condition

The assumption of current practice: connection loads at the facing of a wall were the same as those computed for internal stability design

The connection loads have been evidenced from monitored walls to be the highest loads in a layer of reinforcement



Normalized peak strain values

The cohesive strength component of a backfill soil was often ignored

Internal tensile loads seemed to be excessively over-designed could explain
- connection failures were not systematic in these types of structures
- or good performance many walls even with poor compaction and/or wetted soil due to poor soil surface drainage management.

Development of K-Stiffness Method

- largely empirically based: using back-analysis and curve fitting from full-scale tests

- consider the stiffness of various wall components

- reinforcement strains are prevented from getting large enough to allow failure of the soil ⇒ follow the objective of working stress design method

Allen *et al*. (2003)

reinforcement loads in geosynthetic walls constructed with granular (noncohesive, relatively low silt content)

$$T_{max} = \frac{1}{2} K \gamma (H + S) S_v D_{tmax} \Phi_g \Phi_{local} \Phi_{fs} \Phi_{fb}$$

K = lateral earth pressure coefficient, $K = K_0 = 1 - \sin\phi_{ps}$

$\phi = \phi_{ps}$ = peak plane strain friction angle of the soil

Lade and Lee (1976):

$\phi_{ps} = 1.5\phi_{tx} - 17$ (ϕ_{tx}: peak friction angle from triaxial compression test)

Bolton (1986) and Jewell and Wroth (1987) for dense sand:

$\phi_{ps} = \tan^{-1}(1.2 \tan\phi_{ds})$ (ϕ_{ds}: peak direct shear friction angle)

Development of K-Stiffness Method

Allen *et al*. (2003)

$$T_{max} = \tfrac{1}{2} K \gamma (H + S) S_v D_{tmax} \Phi_g \Phi_{local} \Phi_{fs} \Phi_{fb}$$

γ = unit weight of the soil

H = height of the wall

S = equivalent height of uniform surcharge pressure q (i.e. $S = q/\gamma$)

S_v = tributary area

D_{tmax}: the load distribution factor

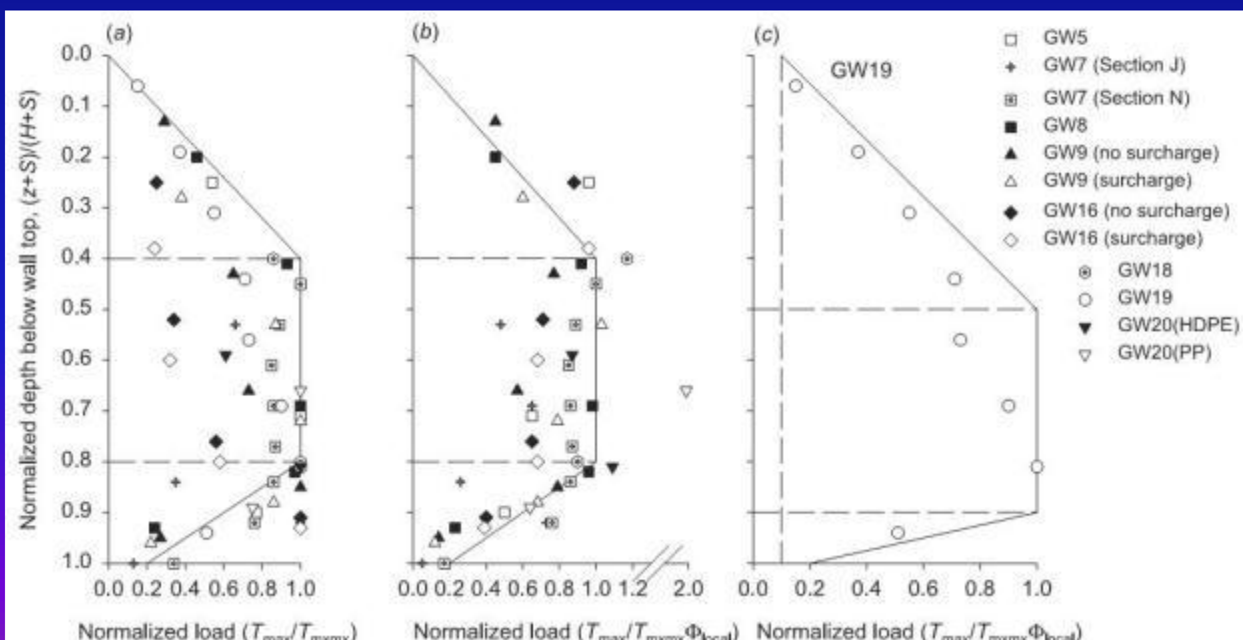


Fig b: better scatter when the local stiffness is considered

Fig c: distribution for polymer strap walls.

Development of K-Stiffness Method

Allen *et al*. (2003)

$T_{max} = \frac{1}{2} K \gamma (H + S) S_v D_{tmax} \Phi_g \Phi_{local} \Phi_{fs} \Phi_{fb}$

Φ_g: global stiffness factor - influence of the stiffness and spacing of the reinforcement layers over the entire wall height

$$\Phi_g = \alpha \left(\frac{S_{global}}{p_a} \right)^{\beta}$$

$$S_{global} = \frac{J_{ave}}{(H/n)} = \frac{\sum_{i=1}^{n} J_i}{H}$$

S_{global} = the global reinforcement stiffness

$\alpha = \beta = 0.25$

p_a = 101 kPa (atmosphere pressure)

J_{ave} = the average tensile stiffness of all n reinforcements

J_i = the tensile stiffness of an individual reinforcement layer

Φ_{local}: local stiffness factor - the relative stiffness of the reinforcement layer with respect to the average stiffness of all reinforcement layers

$$\Phi_{local} = \left(\frac{S_{local}}{S_{global}} \right)^{a}$$

$$S_{local} = \left(\frac{J}{S_v} \right)_i$$

a = 1 for geosynthetic reinforced soil walls

S_{local} = the local reinforcement stiffness for reinforcement layer i

Development of K-Stiffness Method

Allen *et al.* (2003)

$T_{max} = \frac{1}{2} K \gamma (H + S) S_v D_{tmax} \Phi_g \Phi_{local} \boldsymbol{\Phi_{fs}} \Phi_{fb}$

Φ_{fs}: Facing stiffness factor

$$\Phi_{fs} = \eta (F_f)^{\kappa}$$

$$F_f = \frac{1.5 H^4 p_a}{E L b^3 (h_{eff} / H)}$$

F_f = facing column stiffness parameter

b = thickness of the facing column

L = unit length of the facing (e.g., L = 1m)

H = height of the facing column

E = elastic modulus of the "equivalent elastic beam" representing the wall face

h_{eff}= the equivalent height of an un-jointed facing column that is 100% efficient in transmitting moment through the height of the facing column

p_a = 101 kPa (atmosphere pressure)

η, κ = coefficient terms of 0.5 and 0.14, respectively

Development of K-Stiffness Method

Allen *et al.* (2003)

$T_{max} = ½ K \gamma (H + S) S_v D_{tmax} \Phi_g \Phi_{local} \Phi_{fs} \Phi_{fb}$

For preliminary design, Φ_{fs} could be taken:

$\Phi_{fs} = 0.35$ for modular block and propped concrete panel faced walls (stiff facings)

$\Phi_{fs} = 0.5$ for incremental precast concrete facings

$\Phi_{fs} = 1$ for other types of wall facings (flexible facings, e.g., wrapped-face, welded wire, or gabion faced)

Φ_{fb}: Facing batter factor

$$\Phi_{fb} = \left(\frac{K_{abh}}{K_{avh}} \right)^d$$

K_{abh} = the horizontal component of active earth pressure coefficient accounting for wall face batter.

K_{avh} = the horizontal component of active earth pressure coefficient (assuming the wall is vertical).

$d = 0.25$

Development of K-Stiffness Method

Allen *et al.* (2004)

steel reinforced soil walls

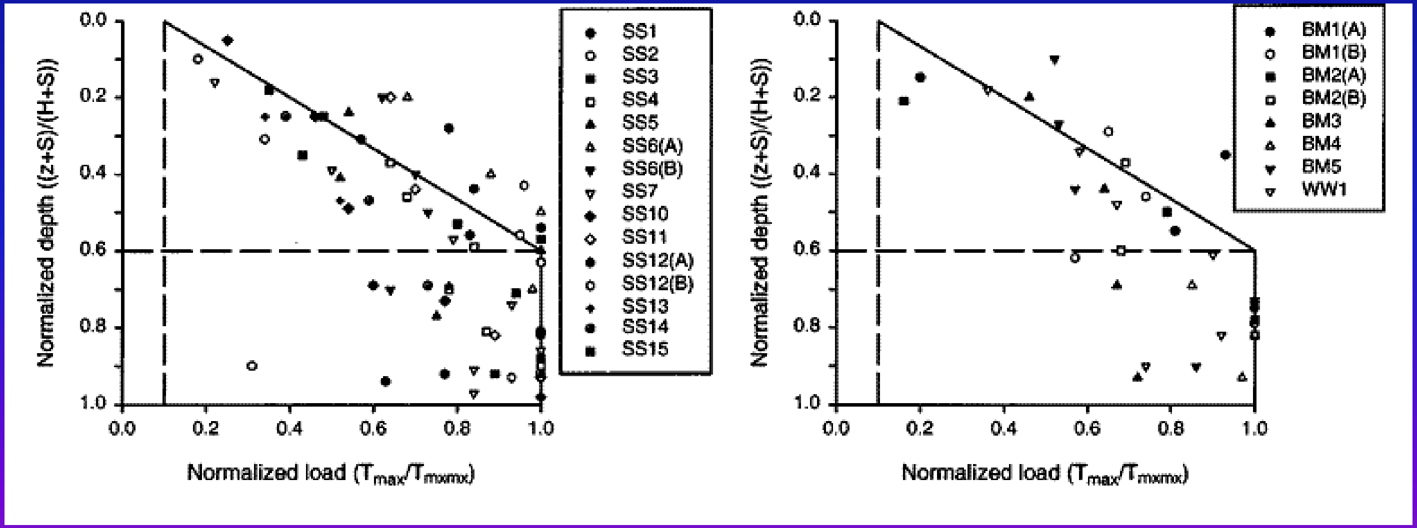
$$T_{max} = \frac{1}{2} K \gamma (H + S) S_v D_{tmax} \Phi_g \Phi_{local} \Phi_{fs} \Phi_{fb}$$

$K = K_0 = 1 - \sin\phi_{ps}$ and $K \geq 0.3$ ($\phi_{ps} = 44^0$) for best correlation between K_0 and T_{max}

$\Phi_{local} = 1$ because a = 0 for steel reinforcement.

Φ_{fs} could be taken as $\Phi_{fs} = 1$

D_{tmax}: Load distribution factor



steel strip

steel bar mat and welded wire

Development of K-Stiffness Method

Miyata and Bathurst (2007a)

re-examine the K-stiffness Method to consider the effect of the facing stiffness factor on the reinforcement loads

$$\Phi_{fs} = \eta (F_f)^{\kappa}$$

Allen *et al*. (2003, 2004)	Miyata and Bathurst (2007)
$\eta = 0.5$ and $\kappa = 0.14$	$\eta = 0.55$ and $\kappa = 0.14$

$\Rightarrow$ better estimation for both geosynthetics and steel reinforced soil walls.

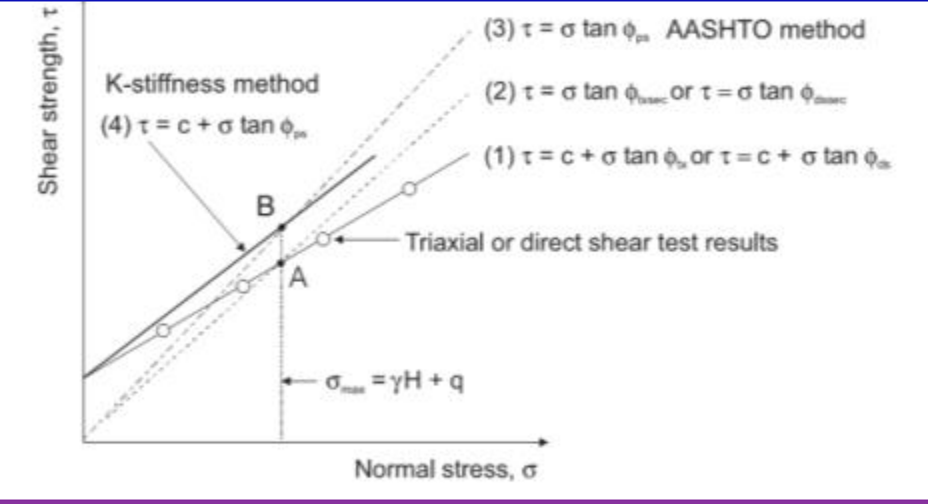
Miyata and Bathurst (2007b)

Geosynthetic reinforced walls

$$T_{max} = \frac{1}{2} K \gamma (H + S) S_v D_{tmax} \Phi_g \Phi_{local} \Phi_{fs} \Phi_{fb} \mathbf{\Phi_c}$$

Φ_c = soil cohesion factor

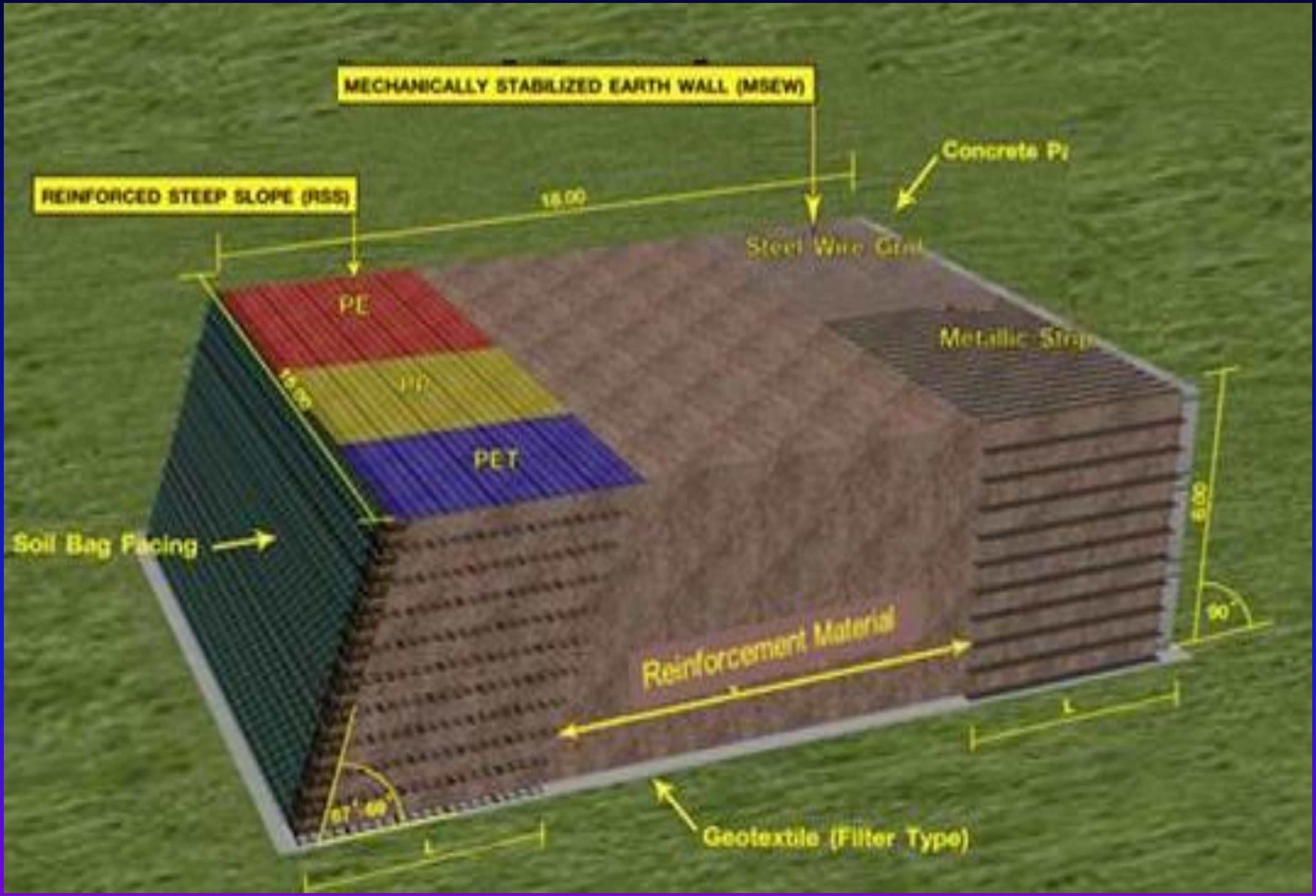
$$\Phi_c = 1 - \lambda \frac{c}{\gamma H}$$



λ = the cohesion coefficient (λ = 6.5)

D_{tmax} = the load distribution factor

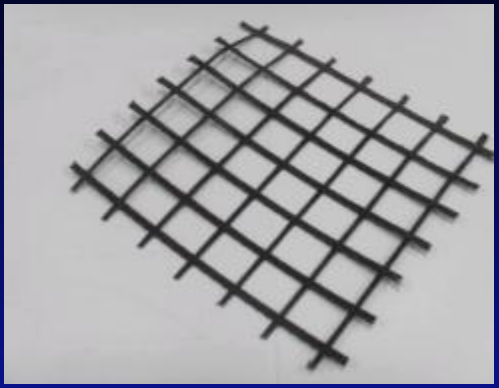
MSE Wall/Embankment



Reinforcing materials

PP

HDPE

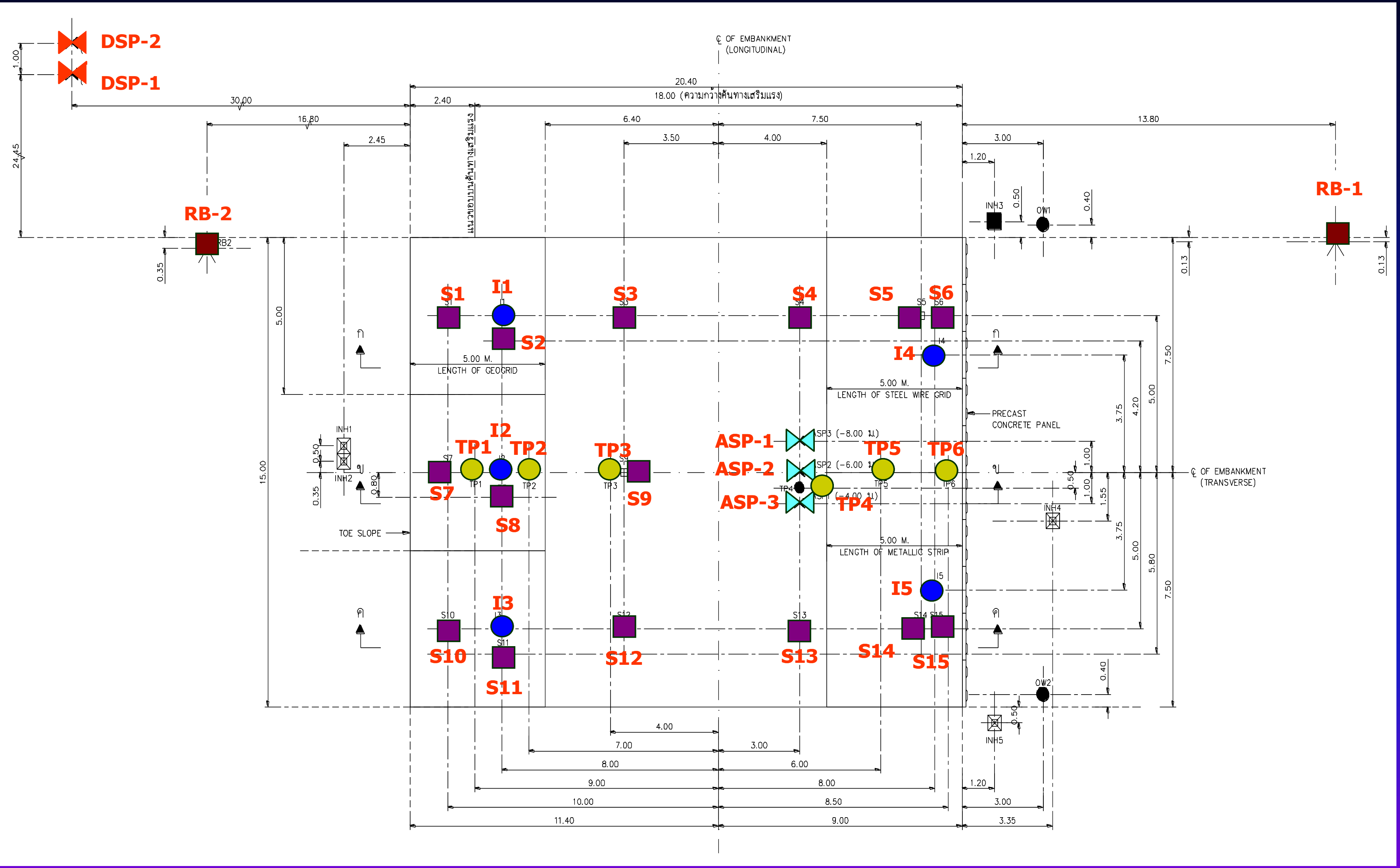


PET

MS

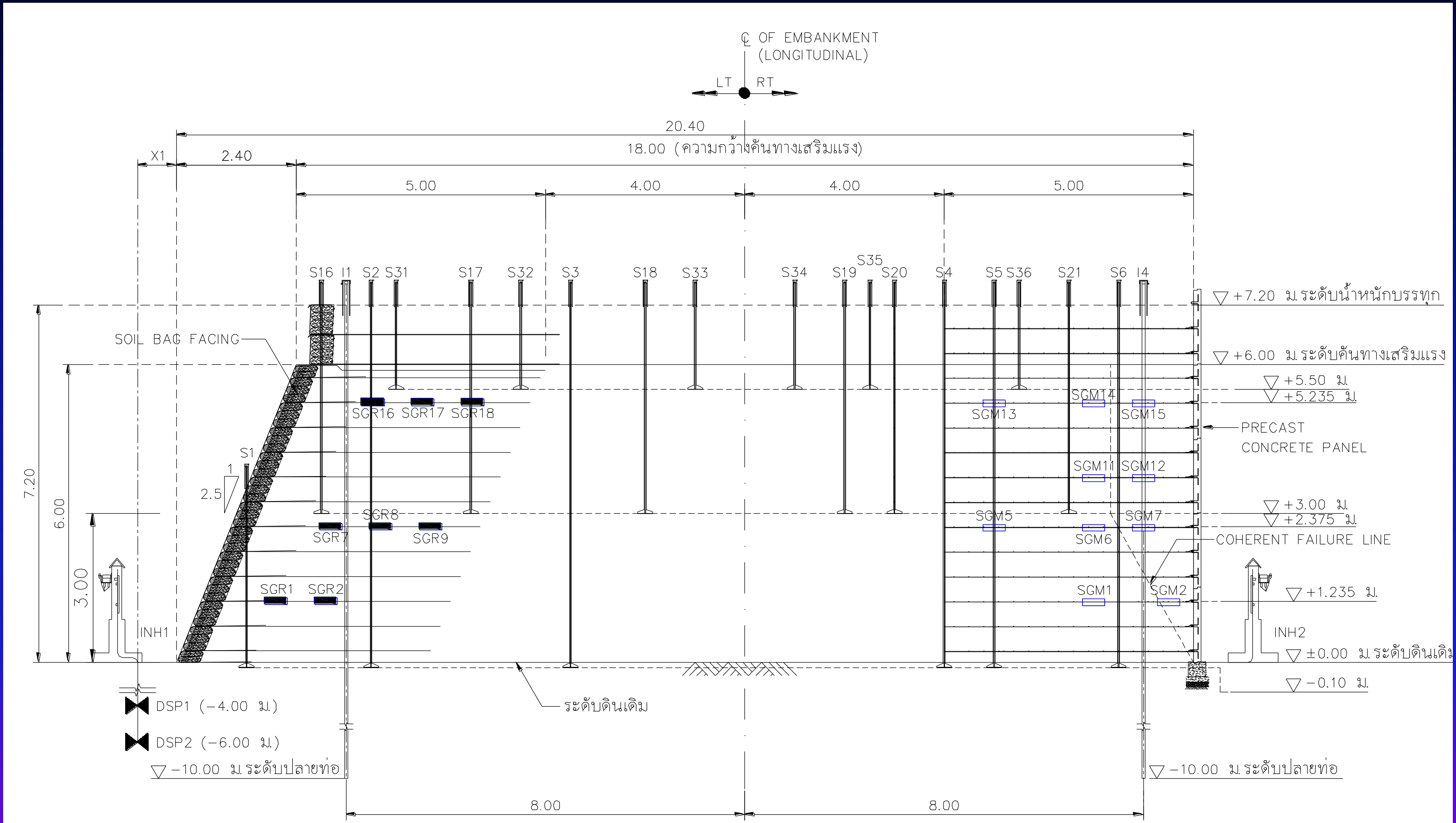
SWG

Instrumentation



Ground Plan

Instrumentation

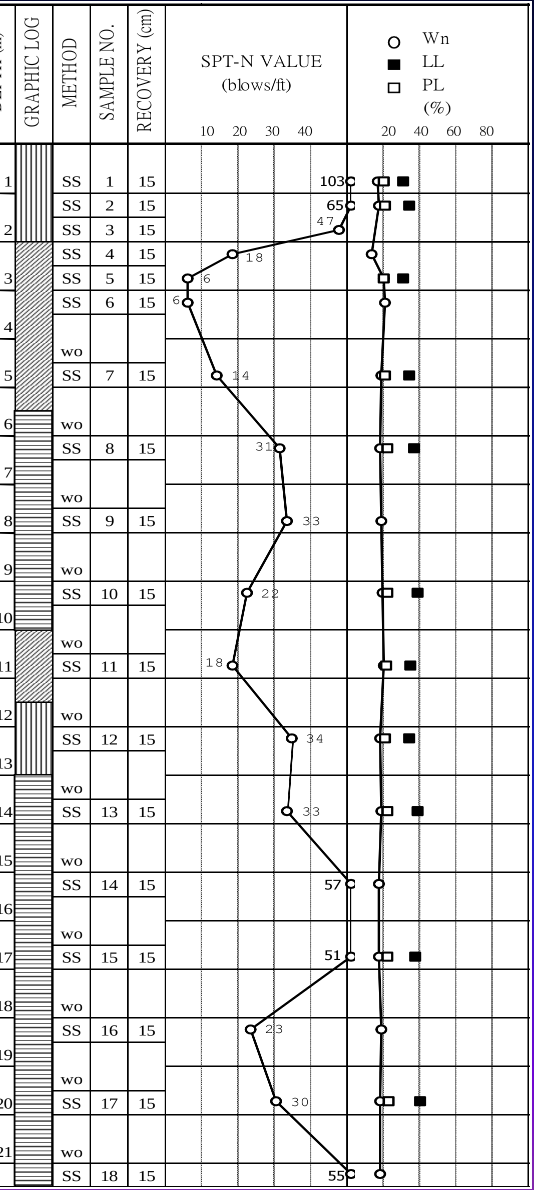


Section

Soil profiles

SOIL DESCRIPTION	Bottom depth (m)
DENSE TO VERY DENSE CLAYEY SAND Fine to Coarse sand, greyish brown (SC)	GWL=2.00
LOOSE TO MEDIUM DENSE CLAYEY SAND Fine to Coarse sand, greyish brown (SC)	5.50
VERY STIFF TO HARD SILTY CLAY Medium plasticity,greyish brown (CL)	10.00
MEDIUM DENSE CLAYEY SAND Fine to Coarse sand, greyish brown (SC)	11.50
DENSE CLAYEY SAND Fine to Coarse sand, greyish brown (SC)	13.00
HARD SILTY CLAY Medium plasticity,greyish brown (CL)	
END OF BORING	21.45

DEPTH (m)	METHOD	SAMPLE NO.	RECOVERY (cm)	SPT-N VALUE (blows/ft)
1	SS	1	15	103
	SS	2	15	65
2	SS	3	15	47
	SS	4	15	18
3	SS	5	15	6
	SS	6	15	6
4				
	wo			
5	SS	7	15	14
6	wo			
	SS	8	15	31
7				
	wo			
8	SS	9	15	33
9	wo			
	SS	10	15	22
10				
	wo			
11	SS	11	15	18
12	wo			
	SS	12	15	34
13				
	wo			
14	SS	13	15	33
15	wo			
	SS	14	15	57
16				
	wo			
17	SS	15	15	51
18	wo			
	SS	16	15	23
19				
	wo			
20	SS	17	15	30
21	wo			
	SS	18	15	55



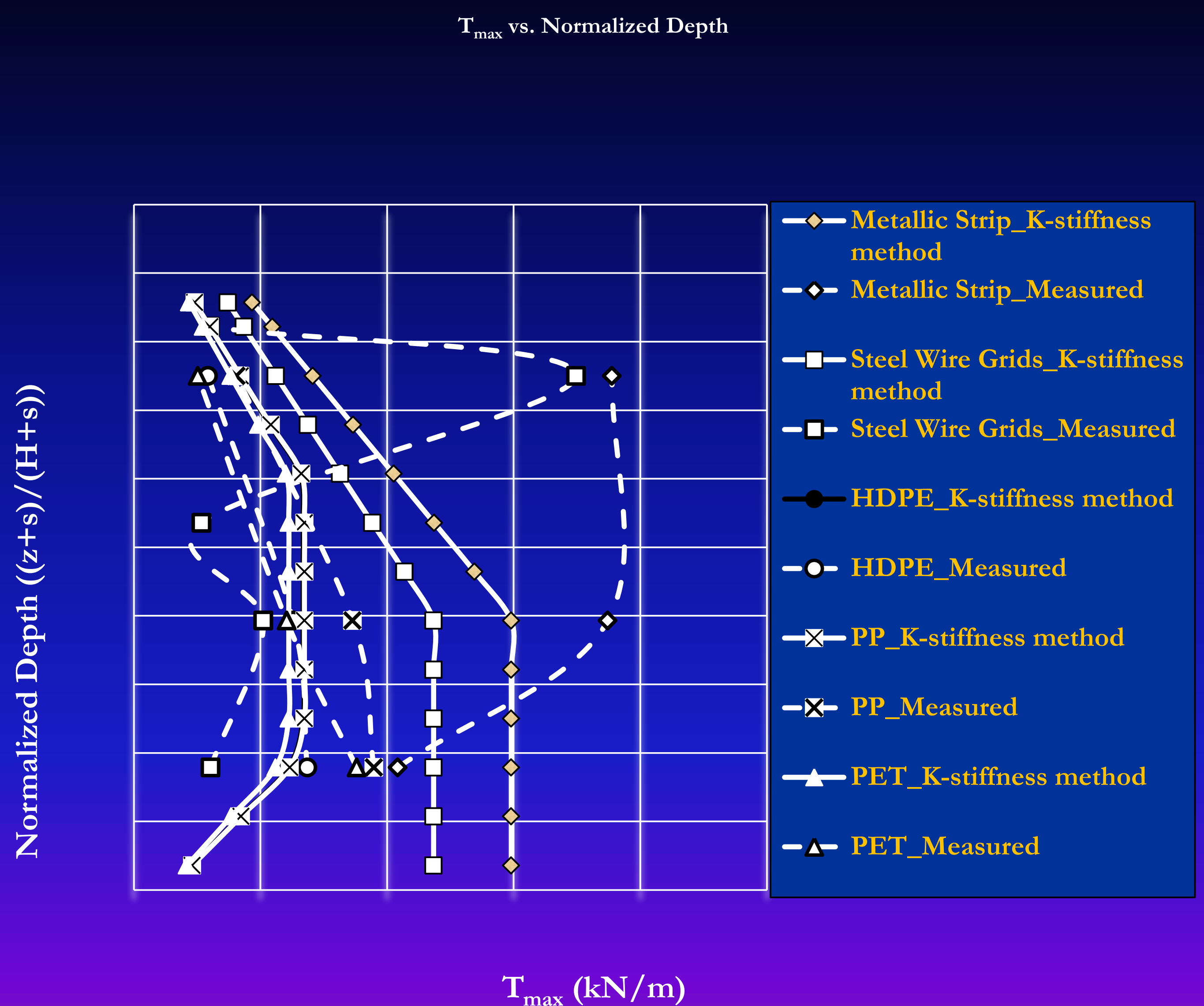
Analyses by the K-stiffness method

$$T_{max} = \frac{1}{2} K \gamma (H + S) S_v D_{tmax} \Phi_g \Phi_{local} \Phi_{fs} \Phi_{fb} \Phi_c$$

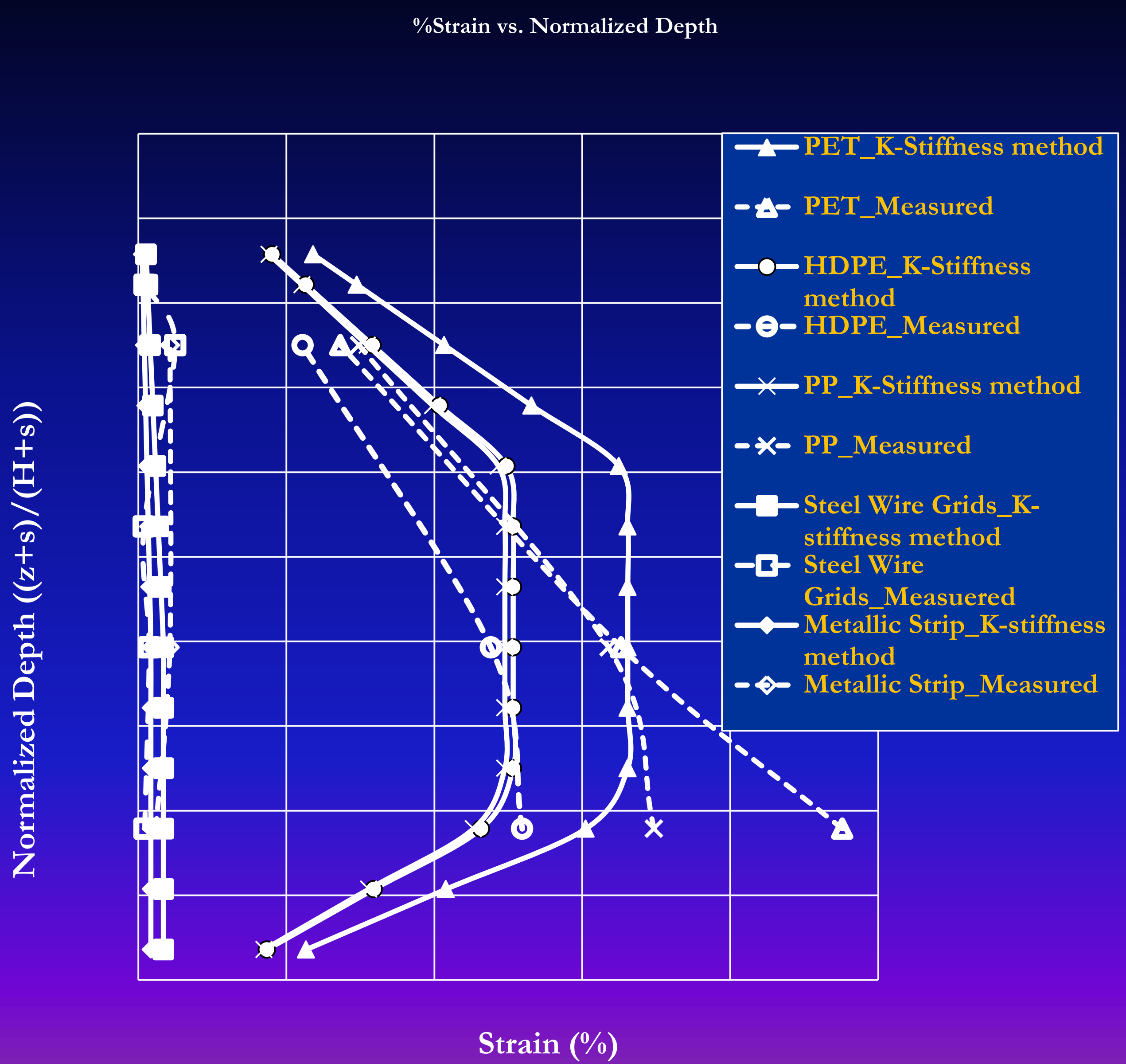
D_{tmax} = the load distribution factor

Φ_g, Φ_{local}, Φ_{fs}, Φ_{fb}, Φ_c are influence factors that account for the effects of global and local reinforcement stiffness, facing stiffness, face batter and soil cohesion, respectively

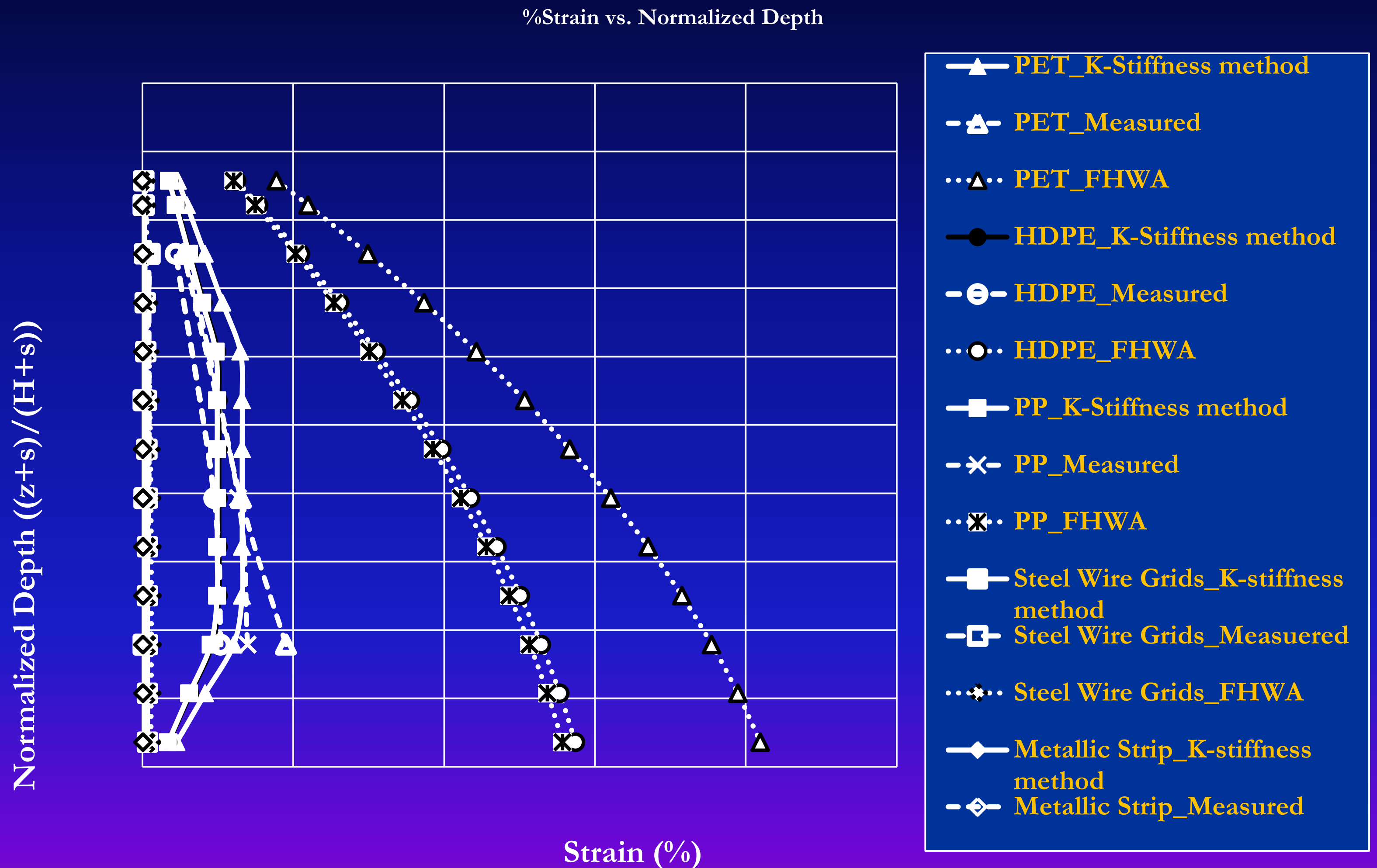
Measurements compared with internal design by the K-stiffness



Measurements compared with internal design by the K-stiffness

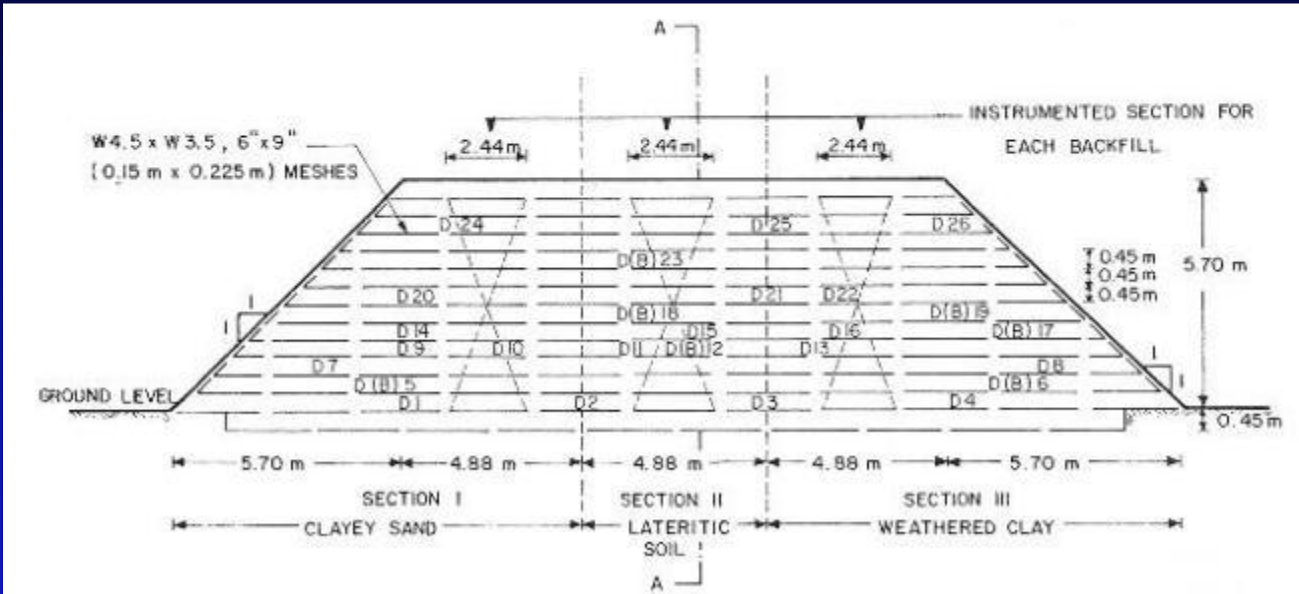


Measurements and internal design by the K-stiffness method compared with internal design by FHWA structure stiffness method



Data obtained from previous studies of MSE structures at AIT Campus on soft ground

Bergado *et al*. (1991)



Front (longitudinal) section of the welded wire wall

Facing: vertical wire mesh

Backfills: Clayey sand
Lateritic soil
Weathered clay

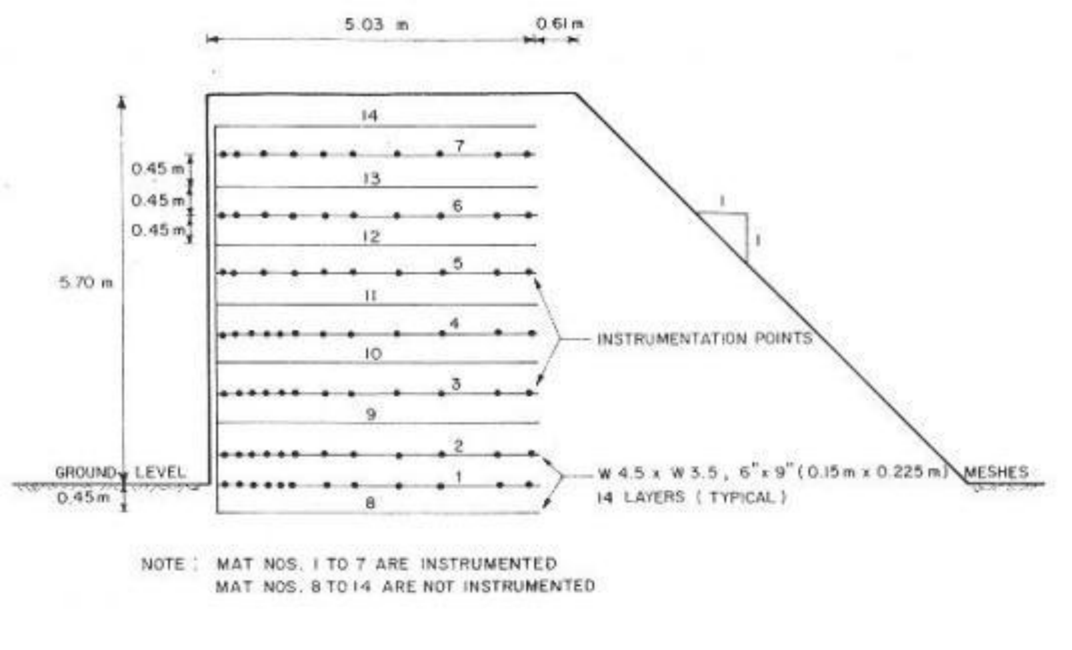
Reinforcement: welded wire mats
2.44 m wide and 5.0 m long, 6 x 9 in. (0.15 x 0.225 m) grid opening

H = 5.7m

L = 14.64m at the top, divided into three sections along its length

Data obtained from previous studies of MSE structures at AIT Campus

Bergado *et al.* (1991)



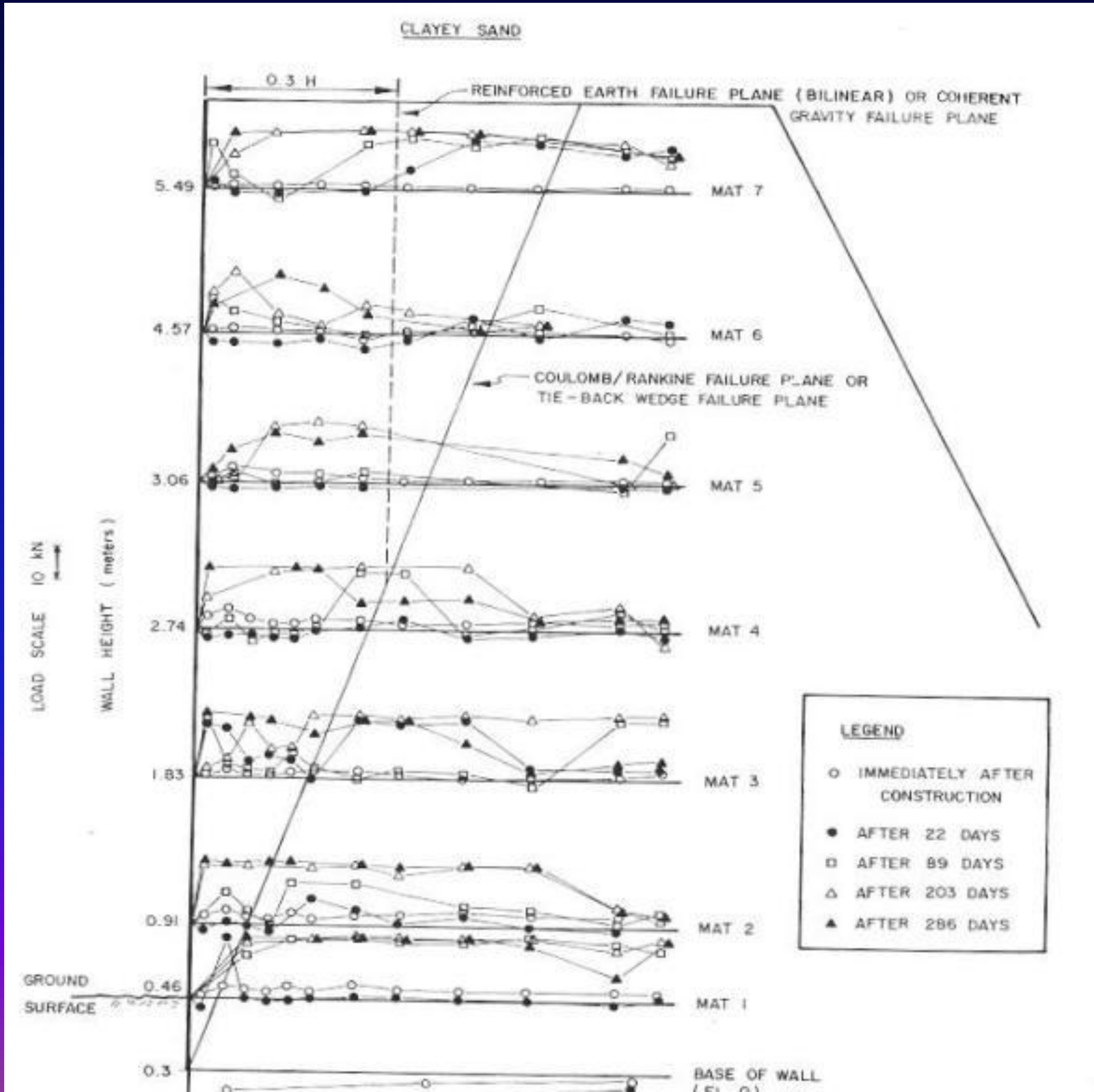
View of the welded wire wall along section A-A

- S_v = 0.45m
- 7 mats instrumented with self-temperature compensating electrical resistant strain gages

Data obtained from previous studies of MSE structures at AIT Campus

Bergado *et al.* (1991)

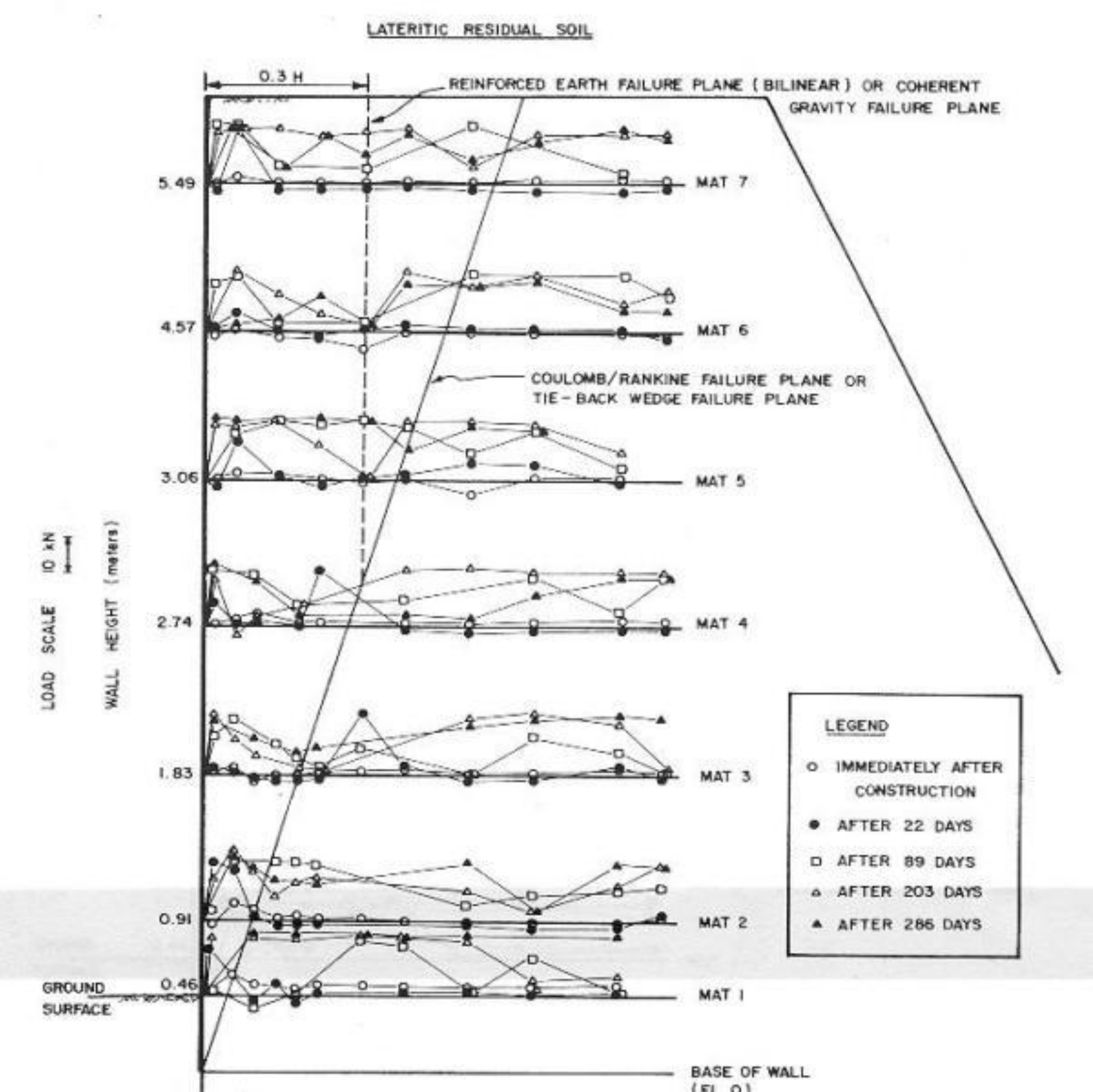
Variation of tensions in the longitudinal bars immediately after construction and for different periods after construction (Clayey sand)



Data obtained from previous studies of MSE structures at AIT Campus

Bergado *et al.* (1991)

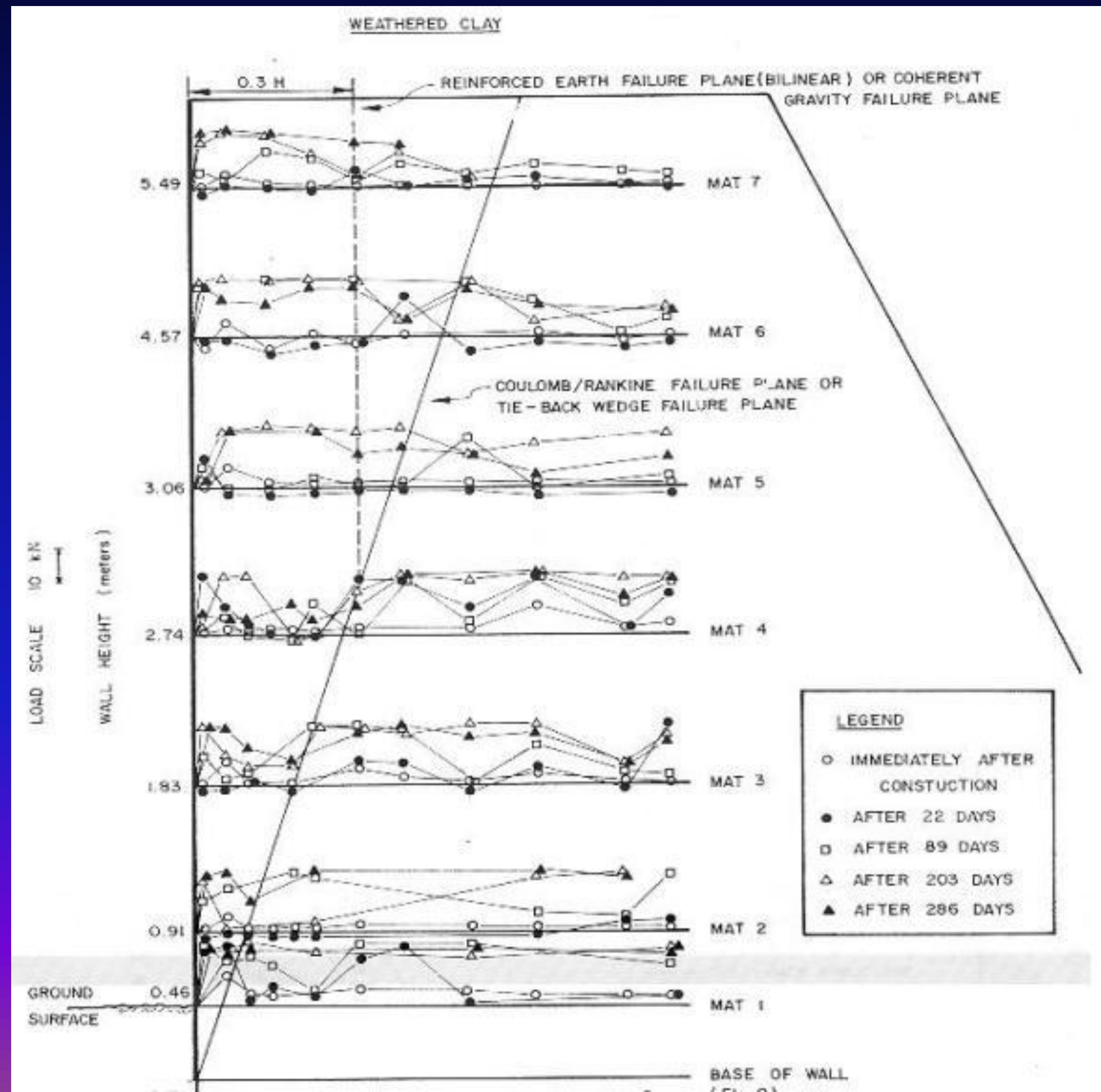
Variation of tensions in the longitudinal bars immediately after construction and for different periods after construction (Lateritic Residual soil)



Data obtained from previous studies of MSE structures at AIT Campus

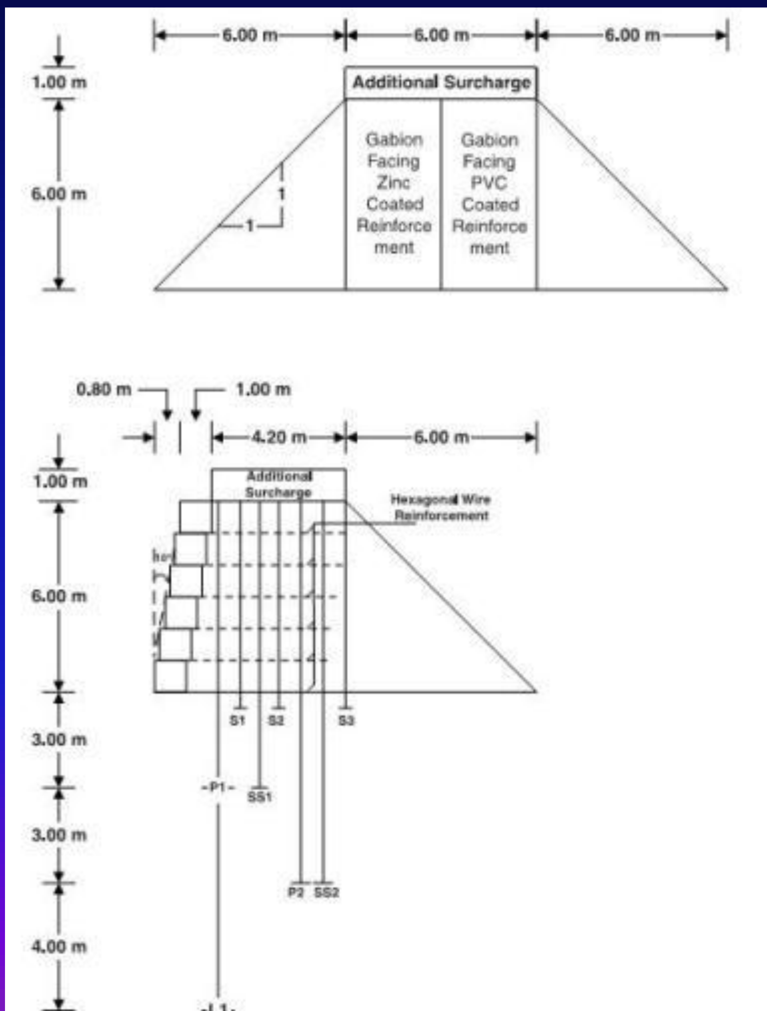
Bergado *et al.* (1991)

Variation of tensions in the longitudinal bars immediately after construction and for different periods after construction (Weathered clay)

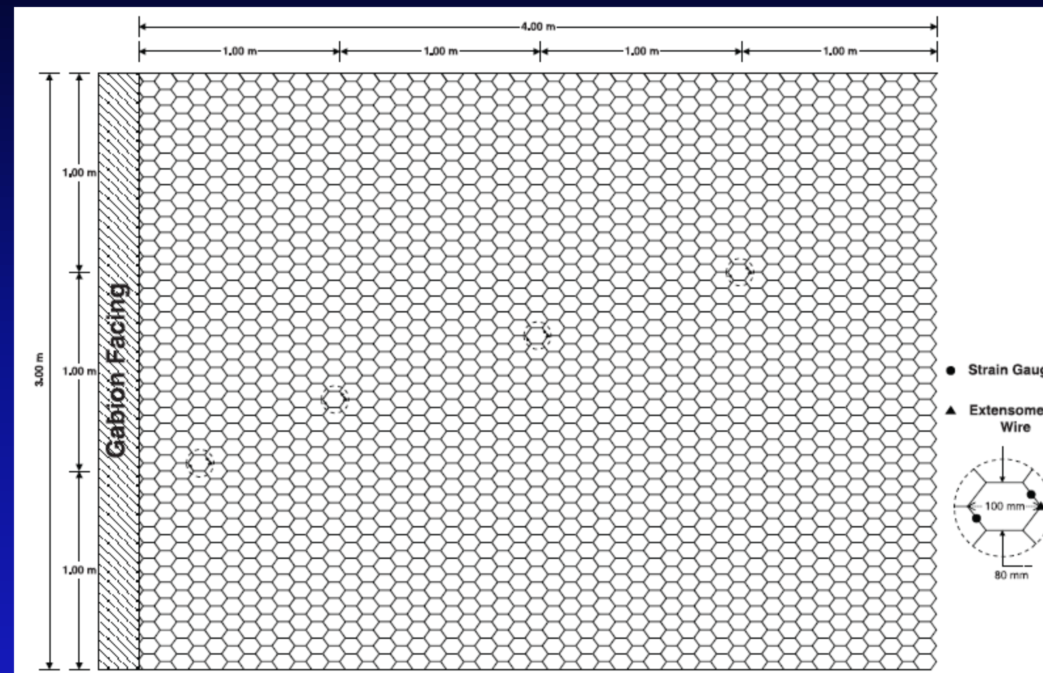


Data obtained from previous studies of MSE structures at AIT Campus

Voottipruex (2000)



Front section and view of the reinforced wall



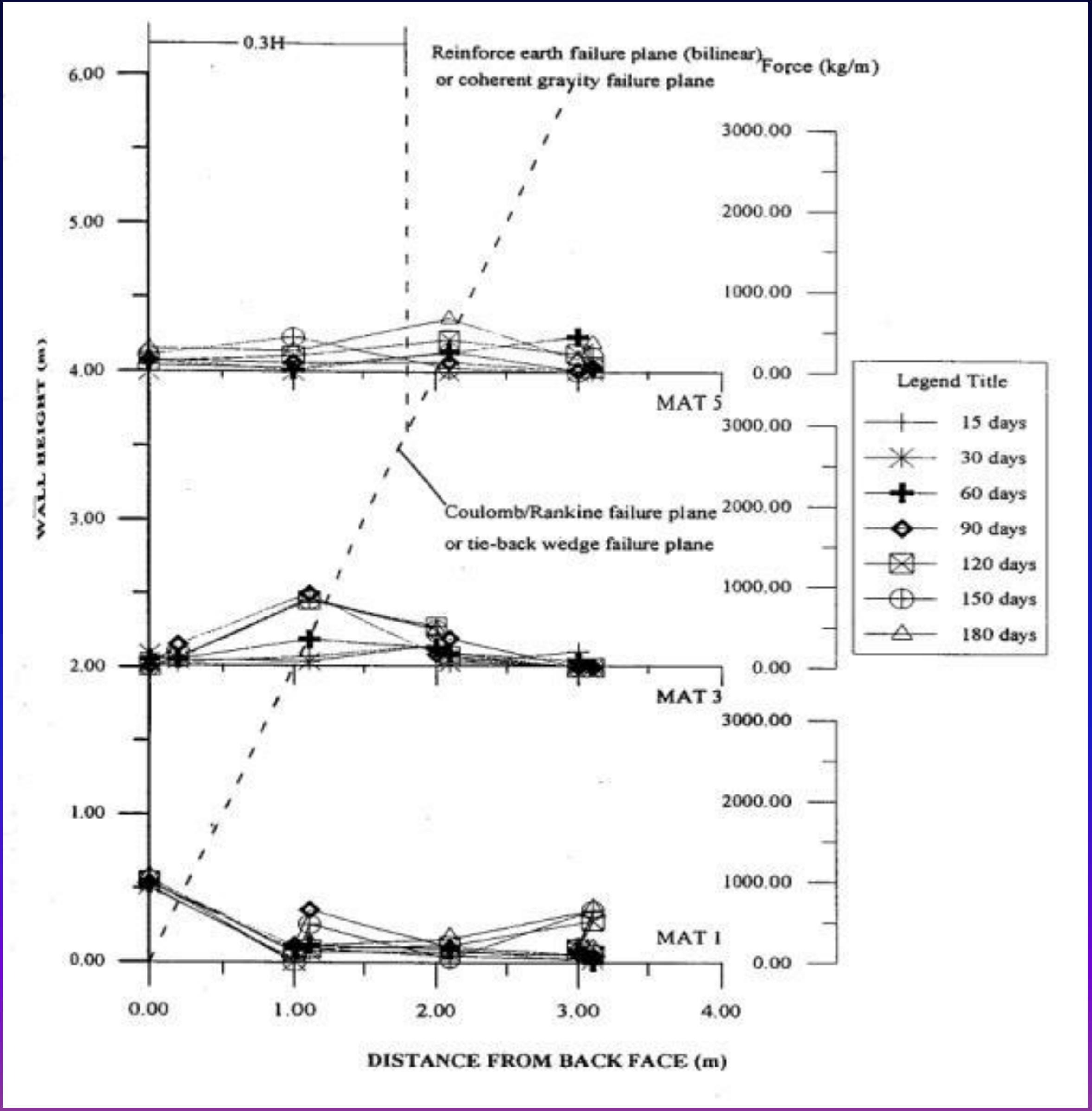
Configuration of hexagonal wire mesh reinforcement

Facing: gabion facing, 10 degree inclined
Reinforcement: hexagonal wire
galvanized coated and PVC-coated
Backfill: silty sand
H = 6m
S_v = 0.5m

Data obtained from previous studies of MSE structures at AIT Campus

Voottipruex (2000)

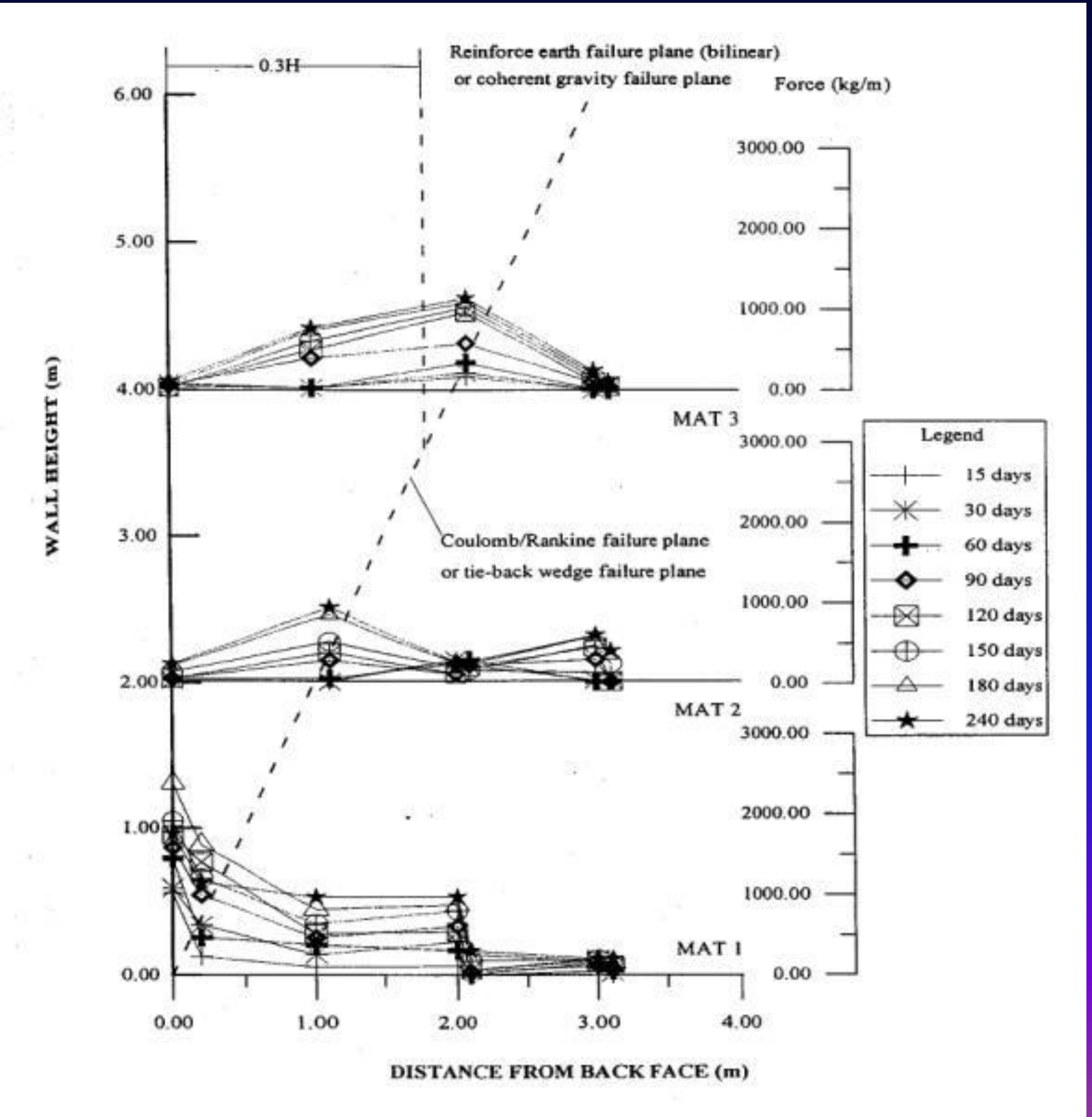
Reinforcement tension of PVC-coated wire mesh in different period after construction



Data obtained from previous studies of MSE structures at AIT Campus

Voottipruex (2000)

Reinforcement tension of zinc-coated wire mesh in different period after construction



Methodology and Results

Modification of
K-stiffness Method

Factors Affecting
The Kinked Steel Grid Reinforcement

Validate the data from previous studies

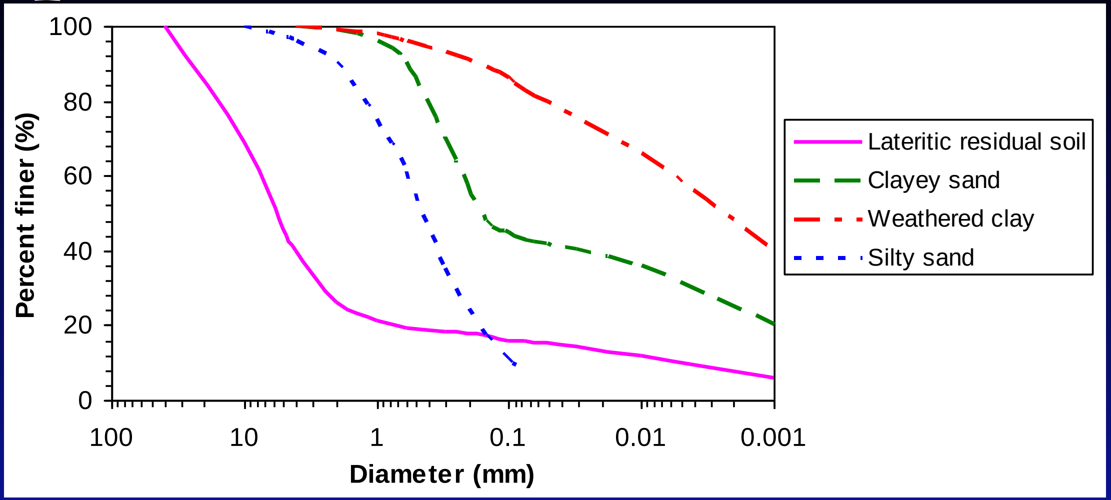
- Simplified method (AASHTO, 2002)
- FHWA Structure stiffness method
- Original K-stiffness method (Allen et al., 2004)
- Modified K-stiffness method (Miyata and Bathurst, 2007b)

Embankments of

Bergado *et al*. (1991)
Voottipruex (2000)

- evaluate the data
- Comments
- Modify these data by K-Stiffness Method

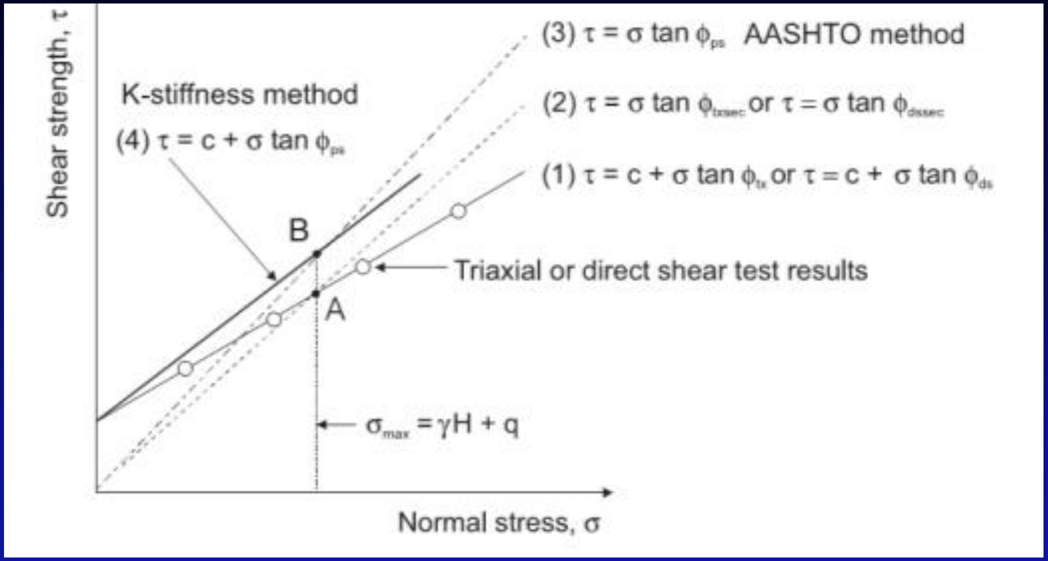
Properties of two embankments



Grain Size distribution of backfill material

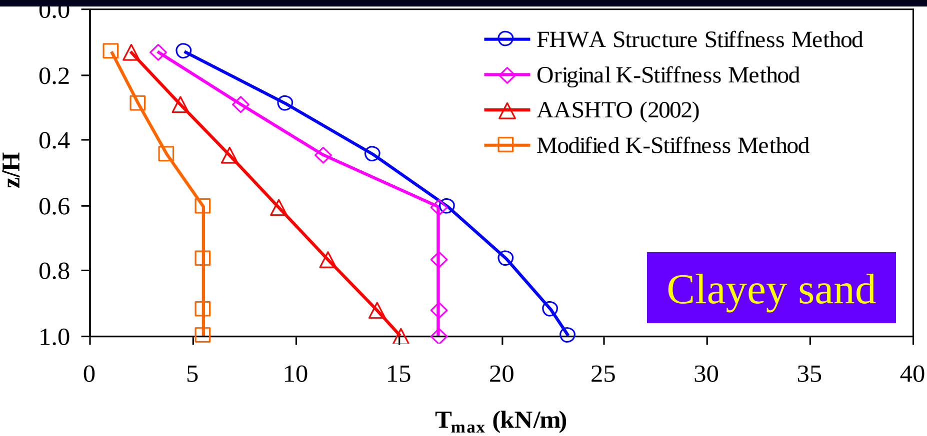
	Bergado et al. (1991)			Voottipruex (2000)	
	Clayey sand	Lateritic residual soil	Weathered clay	Gavalnized coated wire mesh	PVC - coated wire mesh
F_{tx} (^{0})	24	25.2	24	30	30
c (kN/m^2)	10	20	30	5	5
g(kN/m^3)	17	19.3	16.3	18	18
H(m)	5.7	5.7	5.7	6	6
S_v (m)	0.45	0.45	0.45	0.5	0.5
J_i (kN/m)	36000	36000	36000	2170	1140

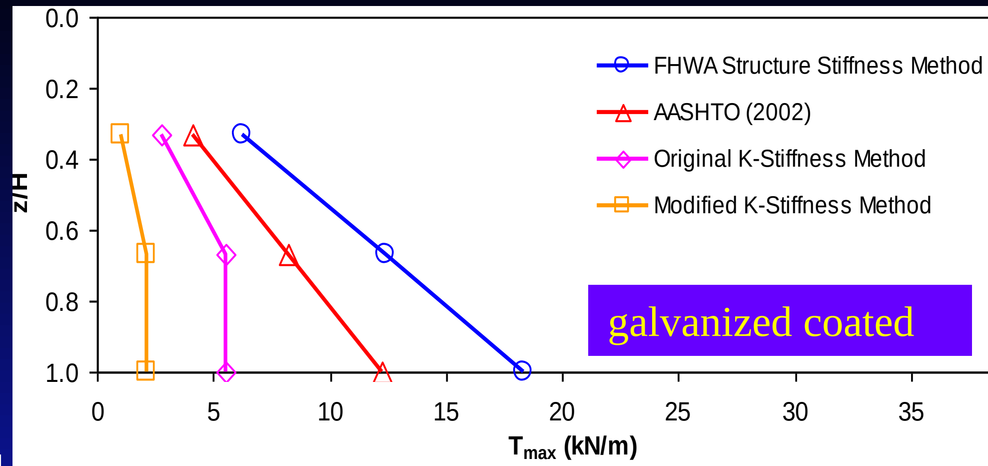
Converted strength parameters

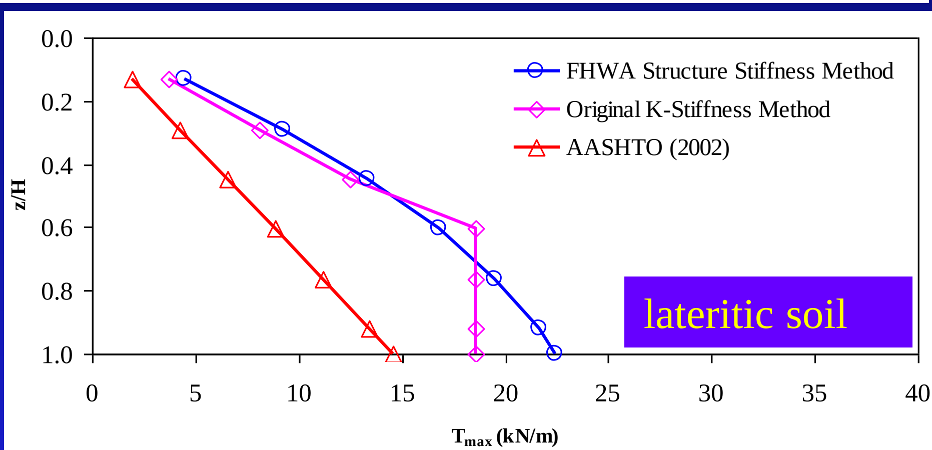


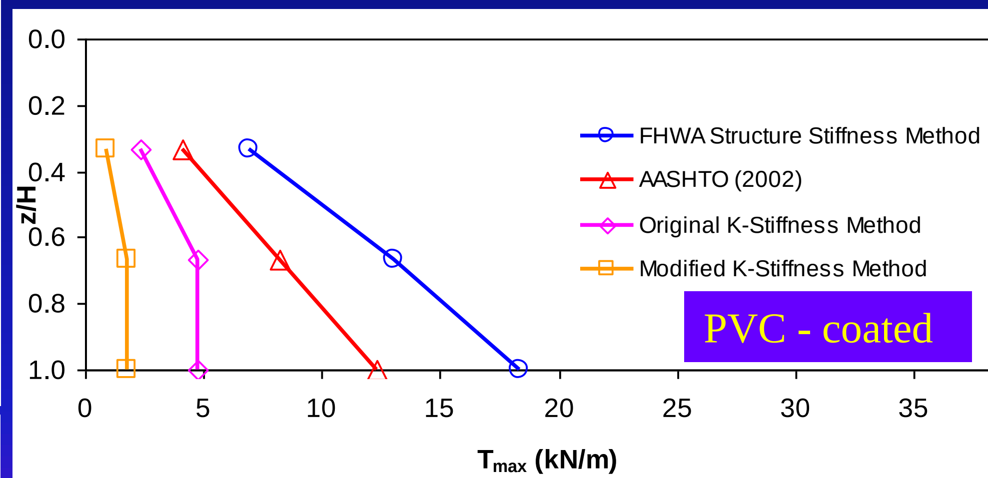
	c (kN/m²)	Φ^0	Φ_{ps} (c = 0)	Φ_{ps} (c > 0)
Silty sand	5	30	39	36
Clayey sand	10	24	29	24
Lateritic residual soil	20	25.2	33	25.2
Weathered clay	30	24	40	27

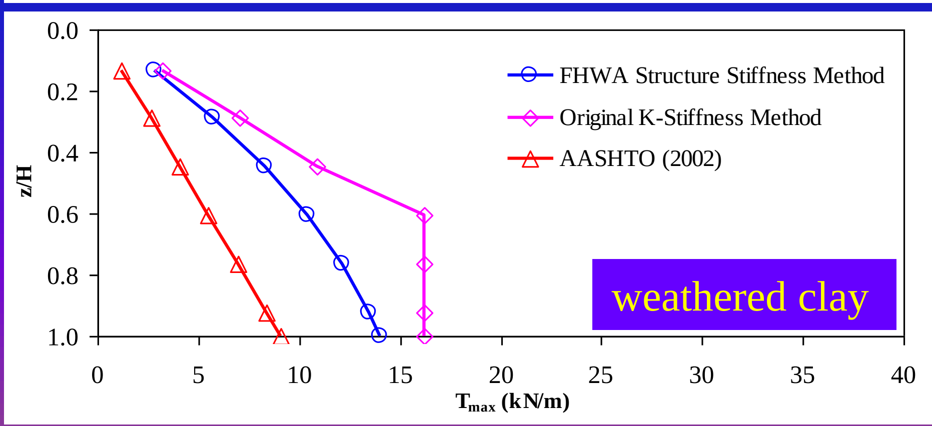
Calculated reinforcement loads







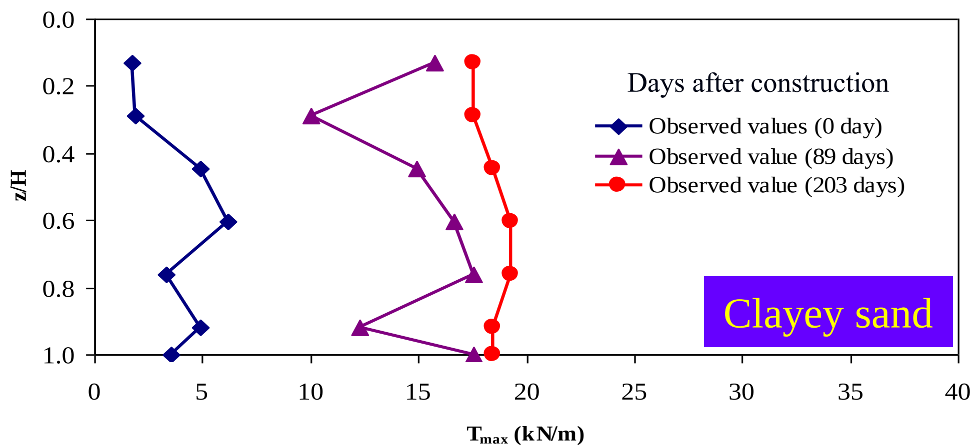


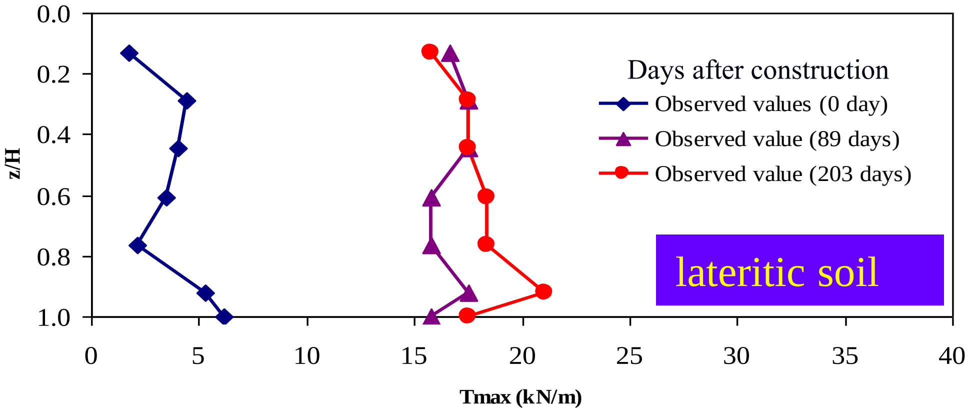


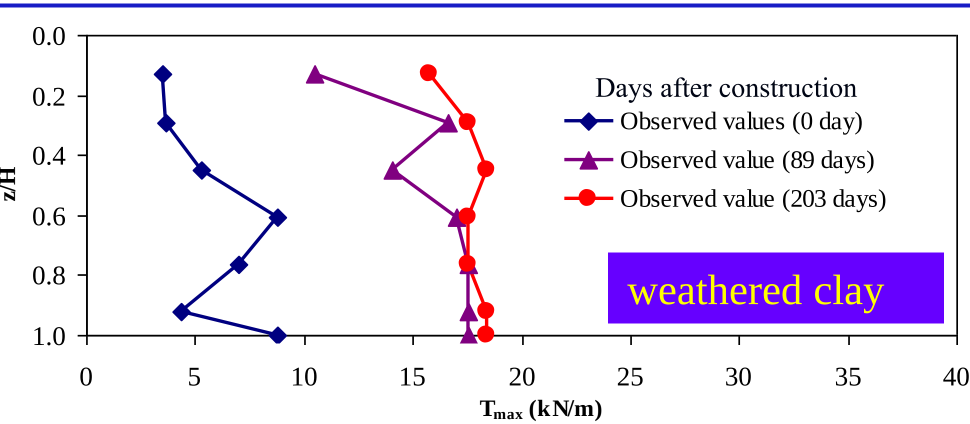
Comments on Results of Validation

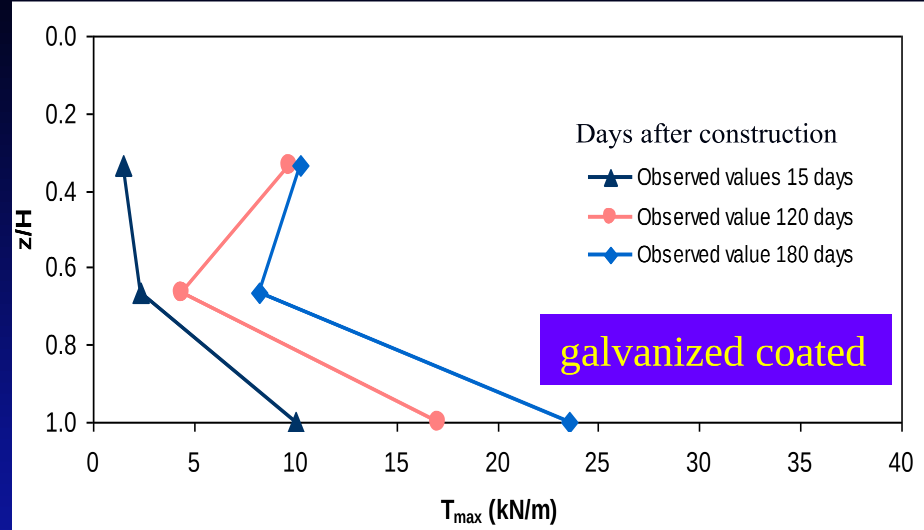
- ✓ Reinforcement loads estimated by FHWA Structure Stiffness Method are 1.5 times higher than those by Simplified Method.
- ✓ Original K-Stiffness Method: suitable for high stiffness steel reinforced structures
  not suitable for the low stiffness steel reinforced structures
- ✓ Modified K-Stiffness Method: much smaller reinforcement load than other approaches
  not applicable for all backfill material with different values of soil cohesion
  - Cannot be applied for steel reinforced walls

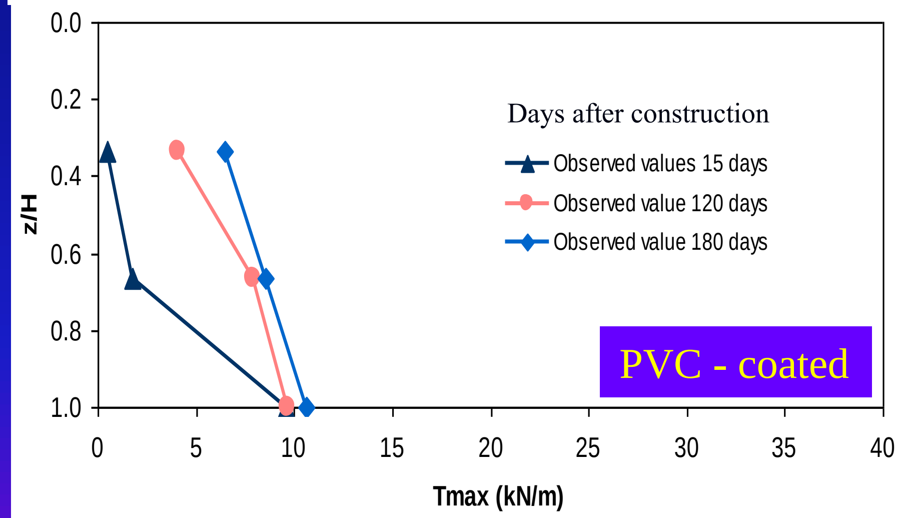
Observed reinforcement loads



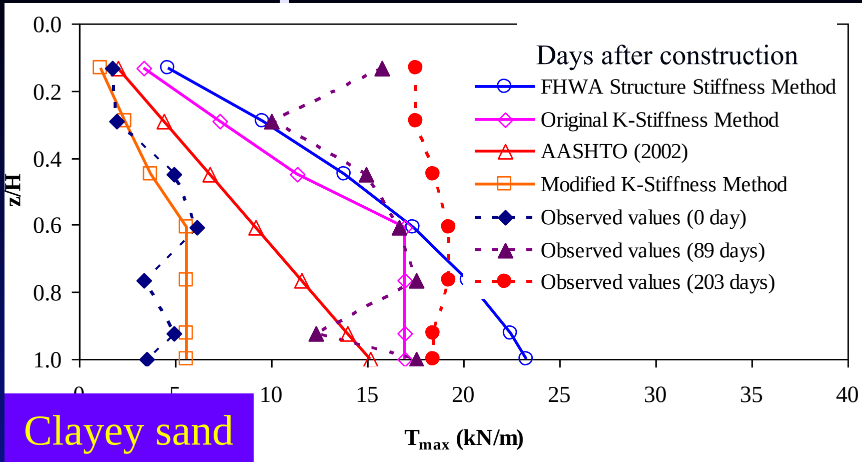


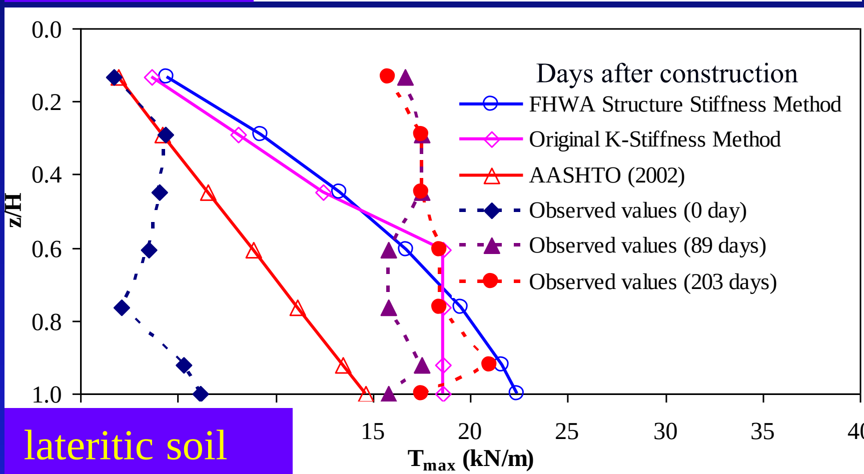


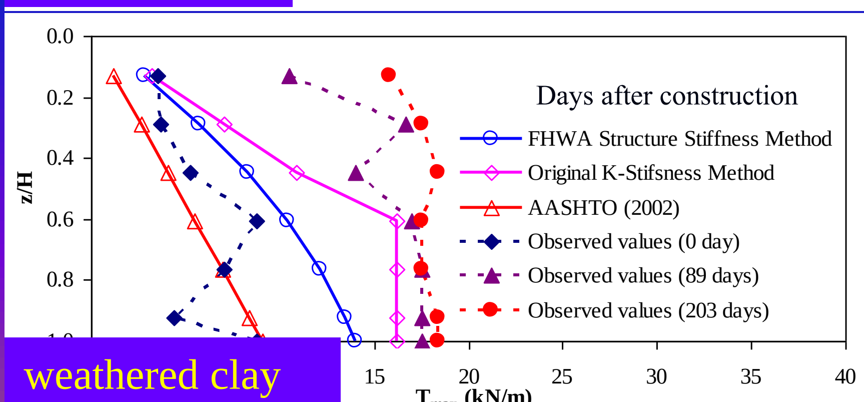


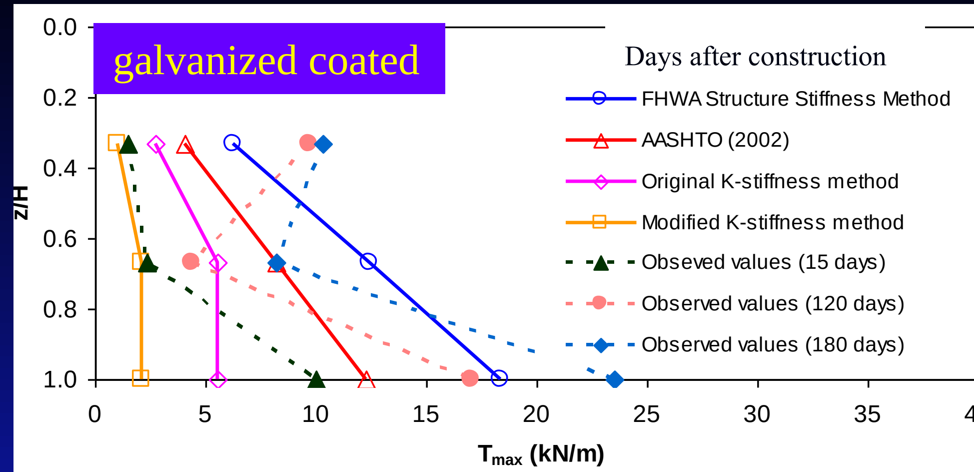


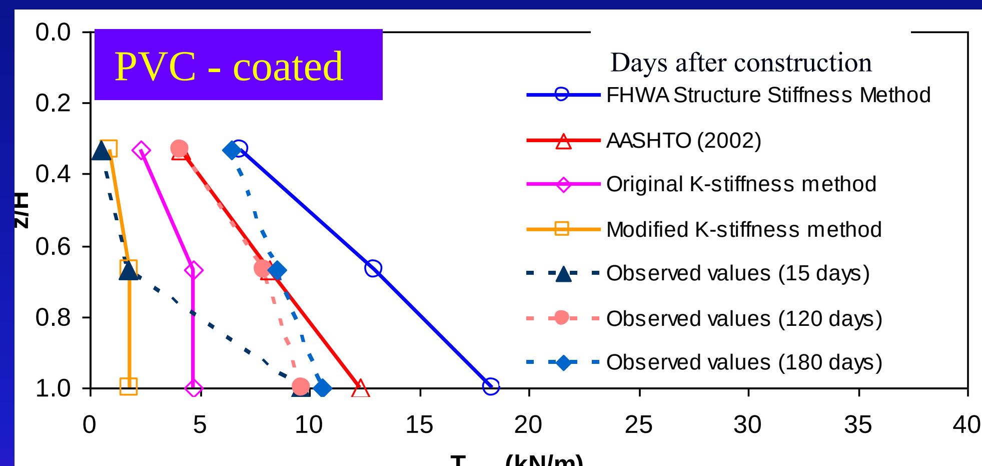
Comparison of calculated and observed values









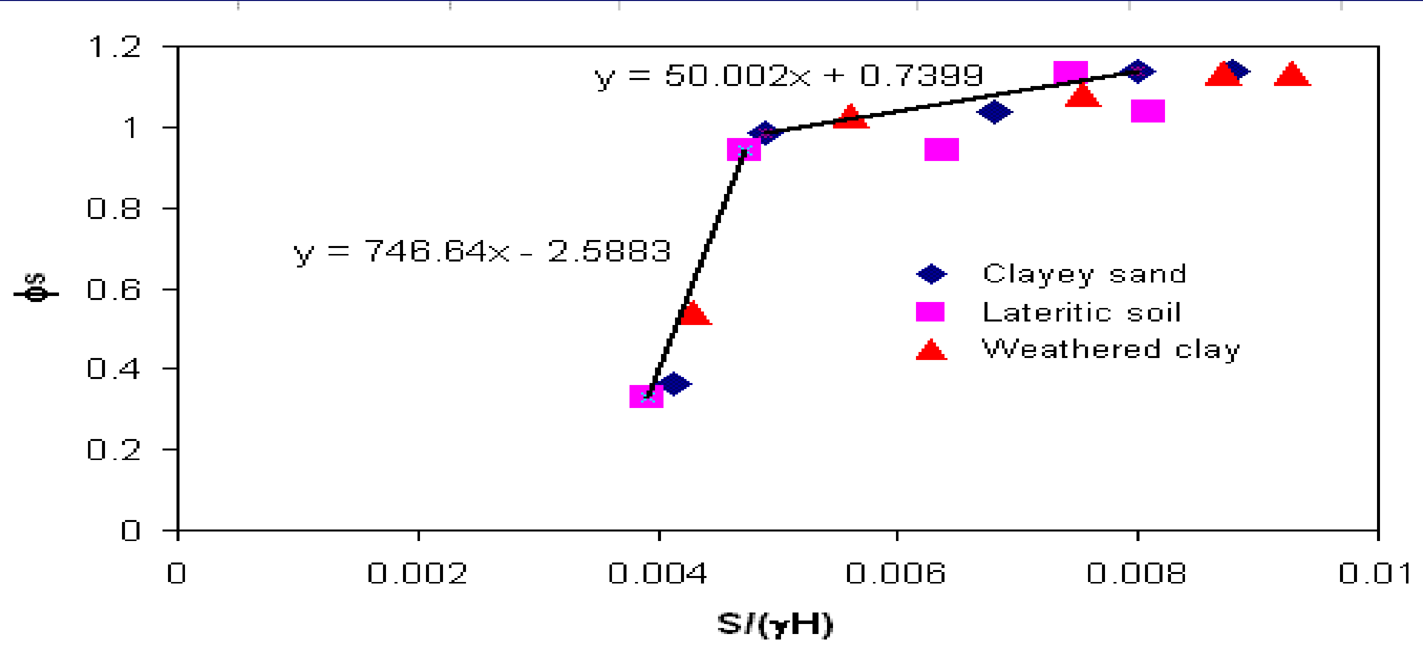


- The State-of-the-Art methods cannot be applied to estimate reinforcement loads for steel reinforced walls constructed on soft foundation.

Modification of Original K-stiffness method

$T_{max} = ½ K Φ (H + S) S_v D_{tmax} Φ_g Φ_{local} Φ_{fs} Φ_{fb} Φ_s$ ($Φ_s$ = the settlement factor)

$$\Phi_s(back - calculated) = \frac{T_{mxmx}(measured)}{½ K \gamma (H+S) S_v \Phi_g \Phi_{local} \Phi_{fb} \Phi_{fs}}$$



If S/gH < 0.005: f_s = 746.64(S/gH) + 2.59

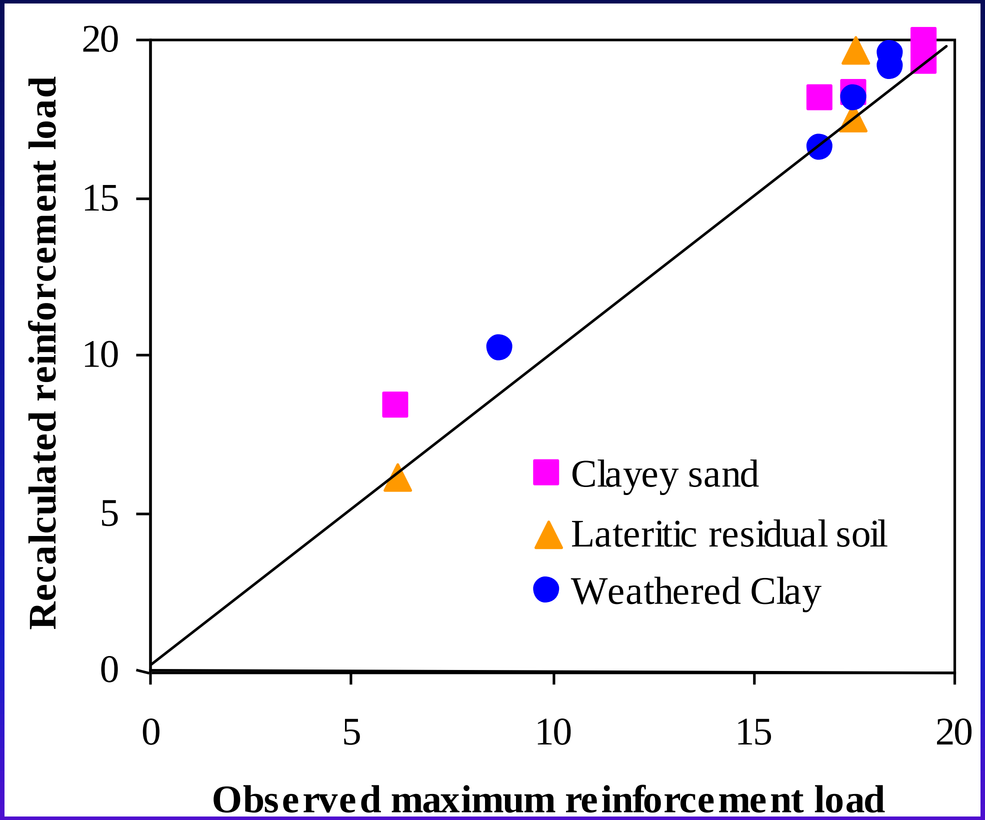
If S/gH > 0.005: f_s = 50(S/gH) – 0.74

Suggestion: D_{tmax} = 1 for 0 < z/H < 1

Validation of modification

Modified original K-stiffness Method:

$$T_{max} = \tfrac{1}{2} K \gamma (H + S) S_v \Phi_g \Phi_{local} \Phi_{fs} \Phi_{fb} \Phi_s$$



Conclusion: Modification of K-Stiffness Method can be applied to estimate the reinforcement loads for steel reinforced structures constructed on soft ground

Thank You !